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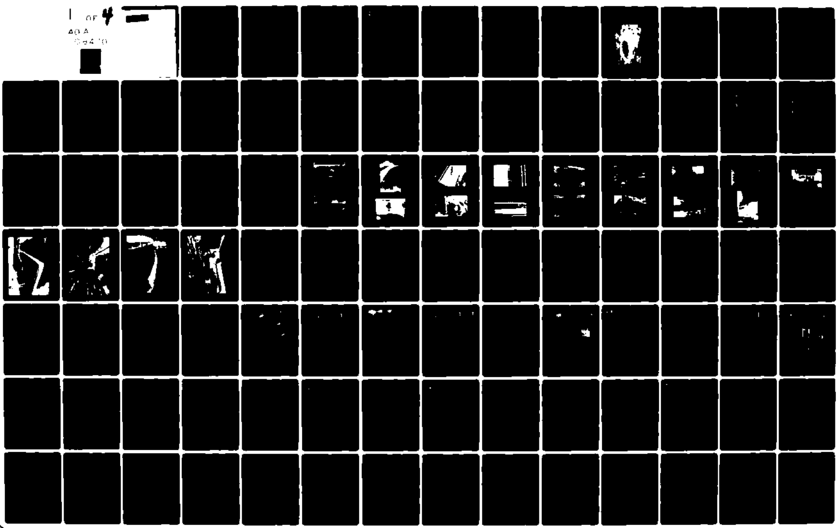

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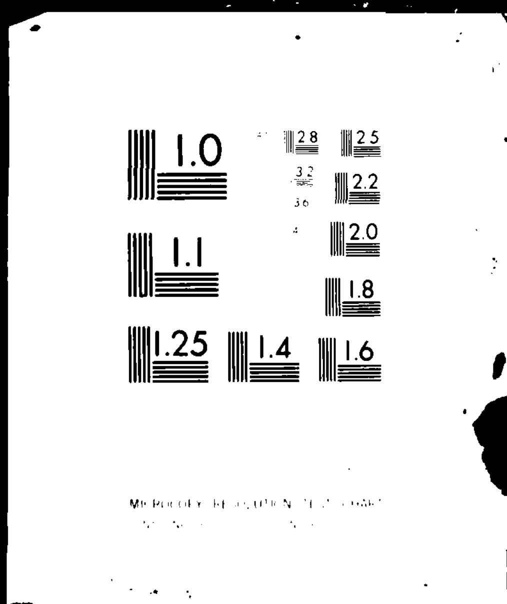

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Visual Inspection
Structural Analysis

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The 8th Avenue Reservoir is a stone masonry structure, constructed in 1889 to be used as a water supply for Nashville. It is elliptical in shape with the minor and major axes being 463 and 603 feet respectively. A dividing wall located along the minor axis divides the reservoir into two basins, each having a capacity of approximately 25 million gallons. The walls of the reservoir are of the gravity type with a top width of 9 feet, base width of almost 23 feet, and a height of 34 feet. A reinforced concrete ring, 8 feet in height, surrounds nearly all of the reservoir. Anchored in sound rock and extending through the

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reservoir wall to the concrete ring are 226 tendons. A gate house located on the north side of the top of the reservoir, houses the two 36 inch diameter influent and effluent pipes that control the water level of the reservoir, which is normally about 3 feet below the top of the reservoir wall. The outflow pipes for the two 21 inch diameter drains of the two basins are also located in the gate house. Also located on the north side of the structure are the access stairs and elevator for the gate house and the chlorine storage room. 8th Avenue Reservoir is in the high hazard and small size classification and according to OCE guidelines must pass the 1/2 to full PMF. Because the only inflow would be the rainfall itself, the reservoir, with its three feet of freeboard, would be able to contain the full PMP of 29.3 inches. The reservoir appeared to be in good condition and structurally stable. No significant seepage, wet areas, sinkholes or cracks were found on or near the reservoir. Seepage was occuring in the dry well of the gate house and calcium carbonate deposits were observed on the exterior stone of the reservoir wall; however, neither considered serious at this time. Structure is given a condition classification of "deficient" because of the seepage in the gate house well and the minor concrete spalling on the walkway of the reservoir.

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DEPARTMENT OF THE ARMY
NASHVILLE DISTRICT, CORPS OF ENGINEERS
P. O. BOX 1070
NASHVILLE, TENNESSEE 37202

IN REPLY REFER TO
ORNED-G

21 SEP 1981

Honorable Lamar Alexander
Governor of Tennessee
Nashville, TN 37219

Dear Governor Alexander:

Furnished herewith is the Phase I Investigation Report on 8th Avenue Reservoir in Nashville, Tennessee. The report was prepared under the authority and provisions of PL 92-367, the National Dam Inspection Act, dated 8 August 1972.

The report presents details of the field inspection, background information, technical analyses, findings, and recommendations for improving the condition of the structure.

Based upon the inspection and subsequent evaluation, 8th Avenue Reservoir is classified as deficient due to minor seepage through the walls of the dry well of the gate house and minor deterioration of the exterior walls.

The recommendation concerning project modifications to control the seepage through the dry well and others contained in this report should be undertaken in the near future.

Public release of the report and initiation of public statements fall within your prerogative. However, under provisions of the Freedom of Information Act, the Corps of Engineers is required to respond fully to inquiries on information contained in the report and to make it accessible for review on request.

Your assistance in keeping me informed of any further developments will be appreciated.

Sincerely,

Kenneth W. Walley, LTC
FOR LEE W. TUCKER
Colonel, Corps of Engineers
Commander

1 Incl
As stated

CF:
Mr. Robert A. Hunt, Director
Division of Water Resources
4721 Trousdale Drive
Nashville, TN 37220

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Structure .... 8th Avenue Reservoir
County .... Davidson
Inspection Date .... 29 June 1981

This inspection and evaluation was prepared by the Engineering ~~Division~~ of the Nashville District of the Corps of Engineers.

PREPARED BY:

Paul F. Bluhm
PAUL F. BLUHM
Civil Engineer

APPROVED BY:

Timothy McCleskey
TIMOTHY McCLESKEY
Chief, I&I Section

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
TENNESSEE

Name of Structure . . . . 8th Avenue Reservoir

County . . . . Davidson

Inspection Date . . . . 22 June 1981

ABSTRACT

The 8th Avenue Reservoir is a stone masonry structure, constructed in 1889 to be used as a water supply for Nashville. It is elliptical in shape with the minor and major axes being 463 and 603 feet respectively. A dividing wall located along the minor axis divides the reservoir into two basins, each having a capacity of approximately 25 million gallons. The walls of the reservoir are of the gravity type with a top width of 9 feet, base width of almost 23 feet, and a height of 34 feet. A reinforced concrete ring, 8 feet in height, surrounds nearly all of the reservoir. Anchored in sound rock and extending through the reservoir wall to the concrete ring are 226 tendons.

A gate house located on the north side of the top of the reservoir, houses the two 36 inch diameter influent and effluent pipes that control the water level of the reservoir, which is normally about 3 feet below the top of the reservoir wall. The outflow pipes for the two 21 inch diameter drains of the two basins are also located in the gate house. Also located on the north side of the structure are the access stairs and elevator for the gate house and the chlorine storage room.

8th Avenue Reservoir is in the high hazard and small size classification and according to OCE guidelines must pass the ½ to full PMF. Because the only inflow would be the rainfall itself, the reservoir, with its three feet of freeboard, would be able to contain the full PMP of 29.3 inches.

The reservoir appeared to be in good condition and structurally stable. No significant seepage, wet areas, sinkholes or cracks were found on or near the reservoir. Seepage was occuring in the dry well of the gate house and calcium carbonate deposits were observed on the exterior stone of the reservoir wall; however, neither were considered serious at this time.

The structure is given a condition classification of "deficient" because of the seepage in the gate house well and the minor concrete spalling on the walkway of the reservoir.

OVERVIEW

8TH. AVENUE RESERVOIR

SECTION 1 - GENERAL

1.1 Authority: The phase I inspection of this dam was conducted under the authority of Tennessee Code Annotated, Section 70-2501 to 70-2530, "The Safe Dams Act of 1973", in cooperation with the U.S. Army Corps of Engineers under the authority of Public Law 92-367, "The National Dam Inspection Act".

1.2 Purpose and Scope: This report is prepared under guidance contained in the Department of the Army, Office of the Chief of Engineers, "Recommended Guidelines for Safety Inspection of Dams", for a Phase I investigation. The purpose of the Phase I investigation is to identify expeditiously those dams which may pose hazard to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed analyses involving topographic mapping, subsurface investigation, testing, and detailed computational evaluations are beyond the scope of Phase I investigations. However, the investigation is intended to identify the need for any such study.

In the review of this report, it should be realized that the reported conditions of the dam are based on observations of field conditions at the time of inspection along with data available to the inspection team. Additional data or data furnished containing incorrect information could alter the findings of this report.

The analyses and the recommendations included in this report are related to the hazard classifications of the structure at the time of this report. Changes in conditions downstream of the dam may change the hazard classification of the structure. A change in hazard classification may in turn change the design flood on which the hydraulic and hydrologic analyses are based and may have a significant impact on assessment of the safety of the structure.

It is important to note that the condition of the dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

1.3 Past Inspections: There have been no past inspections of the 8th Avenue Reservoir.

1.4 Details of Inspection: The Phase I inspection was conducted on June 29 1981. The weather was clear and warm (85°F). The reservoir was at normal pool, elevation 676.5 Inspection team members were:

Nashville Corps of Engineers

Paul Bluhm	Civil Engineer - Inspection Coordinator
Tom Porter	Hydraulic Engineer
Tom Allen	Hydraulic Engineer
Gordon McClellan	Structural Engineer
Randy Bush	Soils Engineer
Wayne Swartz	Geologist

Tennessee Department of Conservation

Bill Culbert	Water Resources Engineer

Metropolitan Government - Department of Water and Sewerage Services

Gene Johnson
Fred Clinard

SECTION 2 - PROJECT DESCRIPTION

2.1 Location: The 8th Avenue Reservoir is located in the city of Nashville, Tennessee, about 1½ miles south of the center of the city. It is bounded by 8th Avenue on the east, Argyle Ave. on the south, 10th Ave. on the west, and Vernon Ave. on the north. The reservoir is shown on the U.S. Geological Survey 7.5 minute Nashville West Quadrangle Map at latitude 36° 08' 18"N and longitude 86° 46' 54" W. Location maps are provided in Appendix B of this report.

2.2 History of the Project: The 8th Avenue Reservoir was constructed in 1889 for the purpose of suppling water for the city of Nashville. It is a masonry structure, oval in shape with a wall in the middle dividing it into the east and west basins. Although a massive structure, it was evident that the design or construction may have been poor as the structure leaked considerably from the time it was built. The leakage increased until November 5, 1912 when a 200 foot segment of the east basin wall failed, flooding the area below. Considerable damage was done, but fortunately no lives were lost. It is assumed that the cause of failure was foundation failure or slippage which caused the wall to move laterally outward. Following the failure, six test pits were excavated around the base of the west wall to examine the foundation for "leakage and weak places". One weak spot was found and replaced with concrete. The wall was then rebuilt in 1914 along with the installation of a perimeter french drain system located adjacent to and inside the east basin wall. In addition, the interior walls of the basins were shotcreted and the floors treated with asphalt and overlaid with concrete.

The reservoir did not have any further problems until 1920 when a horizontal crack was detected on the interior wall of the east basin. This crack was in a section that was adjacent to the section that failed in 1912 and was located about 26 feet below the top of the

wall and extending 120 feet to north from the east end of the major axis. Because of this crack, exploratory holes were drilled and test pits excavated to obtain information about the foundation. As a result of this exploration, portions of the east basin wall were underpinned. In addition, the interior walls of both basins were scaled and shotcreted again and a new concrete floor was laid over the old.

There were no major problems with the reservoir following the 1920 work until May 1975 when seepage was observed on the northwest wall of the west basin. The seepage was near the base of the structure and was greater than had previously been observed. Because of this seepage, an extensive subsurface investigative program was begun which included 48 cored holes at various locations around the base of the reservoir. In addition, 14 inclinometers, 20 extensometers and 7 piezometers were installed to monitor horizontal movements and static water levels. Four test pits were also excavated on the northwest side of the structure to gain additional information.

Based on the information obtained from this exploratory program, it was determined that the subsurface was inadequate to provide a sufficient factor of safety against sliding. To solve this problem, a reinforced concrete ring wall, 8 feet in height, was constructed around most of the base of the reservoir. Tendons anchored in sound rock were installed through this ringwall. This ring surrounded all of the reservoir with the exception of the 1914 rebuilt section and the area north of the major axis that was underpinned in 1920. No remedial work was done in these areas. At this same time, the covers of the reservoir were converted to liners and new covers were installed. In addition to installing the tendons in 1977, a guard room, chlorine cylinder storage room, stairs and elevator were constructed on the north side of the structure.

No additional work has been performed on the reservoir since 1977 and there have not been any additional problems.

2.3 Size and Hazard Classification: According to OCE guidelines, the structure is in the small size category with a height of 34 feet and a storage capacity of 153 acre - feet. The structure is classified in the high hazard potenial category because of the apartment complexes present on the north and west sides of the structure, a park on the south side and a major city thoroughfare on the east side. Should the structure fail at any point around its circumference, substantial damage and loss of life would occur.

2.4 Description of Structure and Appurtenances

2.4.1 Geology: Nashville is located in the Central Basin of Tennessee which is a gentle, sloping, lowland. This Basin is situated on the Nashville Dome, a structural arch with the apex centered near Murfreesboro, Tennessee, and the strata dipping away from the center in all directions. Jointing is generally a conspicuous structural feature of the near horizontal to gently folded bedrock. Small minor faults do occur within this area, but it is a stable area with respect to earthquakes. Nashville is located on the west flank of the structural dome with the Ordovician age bedrock having a gentle dip to the northwest. The inclinations of the beds are variable and locally reversed by numerous small warps usually accompanied by more intense jointing. The area also contains a number of relatively small, generally normal faults. Both the joints and faults become passageways for water movement and possible dissolution of the limestone.

The area in which the reservoir is founded is underlain by shaly limestones of the Ordovician age Catheys Formation. The upper portion of this rock has been weathered to varying depths dependent on the factors involved, i.e., proximity of faulting, purity of limestone and availability of water. This bedrock weathering extends about 10 feet down although in some places it's as much as 20 to 25 feet. Three gravity type faults are present under the reservoir but they are considered inactive. Most of the reservoir was founded on the weathered bedrock with the exception of the work performed in 1914 and 1920. In these sections, it appears that the structure was refounded or underpinned in sound bedrock.

2.4.2 Reservoir: The reservoir is elliptical in shape, with the minor and major axes being 463 and 603 feet, respectively. The reservoir is divided into two basins, the east and west basins, by a wall located along the minor axis. A weir 26.5 feet in width, connects the two basins. The reservoir walls are 33.7 feet in height and are of the gravity type with a top width of 9 feet and a bottom width of 22.75 feet. A four foot high brick parapet wall is present on the outer edge of the top of the wall while an iron and aluminum railing provides protection on the inner edge and on both sides of the dividing wall. The outer faces of the wall were constructed of cut masonry stone with the interior filled with a stone rubble. Concrete was then added to fill the voids of the rubble. Two hundred feet of the east basin wall has been replaced and 80 feet was underpinned following the 1912 failure and 1920 work respectively. An 8-foot high, 3-foot thick reinforced concrete ringwall surrounds the entire base of the reservoir, with the exception of the section replaced in 1914 and the

area north of the major axis underpinned in 1920. Inserted at a 45-degree angle through the ringwall are 226 steel tendons, which have a spacing of between 5 and 7.5 feet. The depth of the tendon anchors, which are designed for a load of 160 kips, range from 28 to 75 feet. The water surface of the reservoir is covered with a vinyl cover, the second one to be used. The original cover was converted to a liner in an attempt to waterproof the interior. Two pumps, one for each basin, are located on top of the cover to remove any accumulated rain water. The water elevation of the east basin is normally kept about 3 feet below the top of the walkway.

2.4.3 Gate House, Elevator, Storage Building and Pump House: On the north side of the structure, located between the two basins on the dividing wall is the gate house. It is a structure 55.7 feet in length, 27.3 feet in width and constructed of brick. It houses the various inflow and outflow pipes, valves and drains for the reservoir. Access to the gate house is by either stairs or elevator located on the north side of the structure. Both the stairs and elevator were constructed along with the chlorine cylinder storage room and guard room in 1977. This addition was 22.7 feet in width, 41.3 feet in length, constructed of brick and keyed into the reservoir wall. Located across the access road, north of the reservoir is the Love Circle pump station. Water from the west basin is pumped by this station to various points of distribution. Dimensions of this building are 16.5'x19'.

2.4.4 Inlet and Outlet Works: The gate house is divided into three wells - the influent, effluent and gate or dry wells. Water from the main pumping station located on Omohundro Drive, Nashville, fills the cast basin of the reservoir through a 36-inch diameter pipe located in the influent well. The water level in the east basin is then regulated by this main pumping station. The water level in the west basin is controlled by a weir located in the dividing wall. Water is distributed from the west basin through the effluent well where a 36-inch diameter pipe is located.

2.4.5 Draw Down Facilities: A 21-inch drain is located in each basin to empty the reservoir should the need occur. The 21-inch pipes from the drains meet a 20-inch pipe which empties into the city sewer system. In addition, 12-inch drains are located in the gate, influent, and effluent wells, to either empty them to remove any seepage that has accumulated. Drainage of the reservoir could be accomplished in about one day.

2.4.6 Instrumentation: To monitor horizontal movements in the foundation and surrounding area, 20 extensometers and 10 inclinometers were installed in 1977. The extensometers are anchored in sound rock and extend through the reservoir wall exiting just above the concrete ringwall. Readout of the extensometers can be made manually where they exit from the reservoir wall or electronically in the storage room where a master console for all of the extensometers is located. The inclinometers are located about 10 feet from the reservoir wall at various points around the structure. The depths of these instruments range from 20 to 40 feet. The last readings made on these instruments was in February 1980 and they are currently not being read. Data from these instruments and monuments indicated no movement.

In addition to these instruments, 34 movement plugs were set in the top of the reservoir wall. A Tennessee Highway Department brass marker is set in the walkway on the east side of the reservoir. From this marker, the elevations of the 34 plugs set in the walkway at regular intervals can be determined. Surveys were conducted to determine if any movement of the wall is occurring. The last survey was made in February 1980 and no additional surveys are currently being made.

There are no piezometers present on the reservoir grounds. Core holes that were used to determine water levels have been filled in.

2.4.7 Surrounding Area: A 1V on 3H earth berm that extends from the reservoir wall to the paved access road, surrounds the structure covering the concrete ring wall. This berm covers about 8 feet of the reservoir wall leaving about 25 feet visible for inspection. The paved access road is approximately 20 feet in width and encircles the structure. The area immediately outside of the reservoir is relatively flat, but then slopes down rather steeply. Apartment complexes are located below the structure on the west and north sides, a park to the south and a major city thoroughfare borders the reservoir on the east side.

2.4.8 Security and Monitoring: A guard is present at all times for security and monitoring purposes. This guard visually inspects the exterior periodically and prevents unauthorized personnel from entering the structure. In addition to the guard inspecting the reservoir, personnel from the main water treatment plant also inspect the exterior of the reservoir periodically.

SECTION 3 - FINDINGS

3.1 Visual Findings

3.1.1 Reservoir - Interior: The top and interior of the reservoir appeared to be in good condition with the exceptions of some minor spalling of the concrete which occurred on the walkway on the north side of the structure. The vinyl reservoir cover was also in good condition.

3.1.2 Gate House Stairs, and Storage Room: The gate house is clean and well maintained. The interior walls of the gate well had considerable seepage (1 to 2 gpm) and calcium carbonate deposits had formed on the walls. A drain in the bottom was sufficient to remove this seepage. The pipe and valves appeared to be in good condition although no valves were operated during the inspection. The stairs, elevator and chlorine storage room, all which were constructed in 1977 were in good condition. The pump station, located north of the reservoir was also found to be in good condition.

3.1.3 Reservoir - Exterior: The exterior walls were inspected closely and some signs of deterioration of the masonry stone was apparent although it was not considered serious. A small amount of seepage was observed emerging from the east basin wall (in the section that was replaced in 1914). This seepage was unmeasurable and not considered serious. Calcium carbonate deposits were also observed on this section of the wall and at other points around the reservoir. These deposits indicate that seepage may have been occurring, but again it was not considered serious.

There were no signs of seepage, sinkholes, piping or other signs of distress around the foundation or in the area immediately surrounding the reservoir.

The extensometers and inclinometers were in good condition with the exception of five inclinometers which were missing caps.

3.2 Review of Data: Information available for review included an engineering report on the stability of the 8th Avenue Reservoir prepared by the Chester Engineers, design plans and specifications for the installation of the tendons and construction of the concrete ringwall storage rooms, elevator and stairs, and various correspondences between Geologic Associates and Metropolitan Government. As concluded from Chester Engineer's report and from the available data, the reservoir is founded on "bedrock overlain with clay seams." It is also concluded that prior to the installation of the concrete ringwall and tendons, the foundation was in a marginal state of stability. Installation of the tendons and ringwall has increased the design margin of safety to 1.5. Data from the instrumentation and monumentation installed indicated that no movement of the foundation occurred following completion of the tendons and ringwall.

3.3 Static and Seismic Stability: The actual margin of safety for static stability has been determined based on the data obtained from the exploration programs and design computations. The margin of safety of the structure in its present condition is determined to be 1.5. An assessment of the structural stability based on visual evidence and engineering judgement also indicate a stable structure. The project is located in Seismic Zone I and according to OCE guidelines, should not be expected to be threatened by seismic effects provided static conditions are satisfied.

3.4 Hydraulic and Hydrologic Analysis: According to OCE guidelines, the design flood for a small size dam in a high hazard area is the 1/2 to full Probable Maximum Flood (PMF). The Probable Maximum Precipitation (PMP) for this area is 29.3 inches in six hours. There is no runoff other then the rainfall that accumulates on top of the cover. Because there is a difference of 3 feet between the walkway and cover, the reservoir would not overtop during the full PMP.

3.5 Conclusions and Recommendations:

3.5.1 Conclusions:

a. On the basis of the available data, design calculations, visual evidence and engineering judgement, the reservoir is considered to be structurally stable. The small amount of seepage and calcium carbonate deposits observed on the exterior wall are not considered serious. However, the interior walls of the gate well, of the gate house, did have a significant amount of seepage emerging and deposits forming.

b. The exterior walls show signs of deterioration which is attributed to the poor weathering resistance of the shaly limestone used in the reservoir's construction.

c. The pipes and valves that controlled the water level in the reservoir appeared to be good condition although none of the valves were operated.

d. The spalling on the concrete walkway is not serious.

e. The instrumentation is well protected except for the five inclinometers that are lacking caps.

f. The reservoir will not be overtopped by the full Probable Maximum Flood as required by OCE guidelines for structures of small size and high hazard potential.

g. The project is situated in Seismic Zone 1, indicating that risk of damage from seismic activity is only minor.

h. The reservoir is well protected from vandalism, as a guard is posted to protect the grounds.

i. The reservoir is considered "deficient" due to seepage in the gate well, concrete spalling of the walkway and deterioration of the exterior walls.

3.5.2 Recommendations: The owner should:

a. Repair the areas of the walkway that have been spalling.

b. Waterproof the interior walls of both basins near the gate house to prevent the occurrence of seepage in the gate well.

c. Provide caps for all inclinometers.

d. Establish a program where the instrumentation and monumentation are read on a periodic basis and a record of the data be made readily available for review by a qualified engineer should a serious condition occur.

e. Develop an emergency action plan to alert downstream residents in the event a major problem develops with the reservoir.

f. Continue its periodic inspections of the exterior wall by both the guard and personnel from the sewage treatment plant.

g. Evaluate the structural stability and integrity of the reservoir wall including the effects of the ring wall and tendons on the structure, if it was not previously evaluated in 1975-76.

h. Establish a system to monitor the wall seepage and deterioration.

SECTION 4 REVIEW BOARD FINDINGS

The Interagency Review Board for the National Program of Inspection of Non-Federal Dams met in Nashville on 3 September 1981 to examine the technical data contained in the Phase I investigation report for 8th Avenue Reservoir. The Review Board considered the information and agreed with the report conclusions and recommendations. A copy of the letter report presented by the Review Board is included in Appendix G.

APPENDIX A

DATA SUMMARY

APPENDIX A
DATA SUMMARY SHEET
8TH AVENUE RESERVOIR
DAVIDSON COUNTY, TENNESSEE

A.1 Reservoir

A.1.1 Type: The reservoir is elliptical in shape and divided into two basins by a dividing wall. The walls are of the gravity type with the exterior faces of the wall constructed of cut masonry stone. The interior walls are filled with stone rubble and concrete.

A.1.2 Dimensions, Elevations and Storage Capacity: Elevations are expressed in feet and are referenced to the top of the reservoir which was assumed to be 679.5.

a. Major axis length (interior, at top of wall): 615.54 feet

b. Minor axis length (interior, at top of wall): 475.52 feet

c. Perimeter length (on top of reservoir): 1765 feet

d. Height (from top of ground to top of wall): 24 feet (Ground elevation is about 8-9 feet above base of wall)

e. Ground slope at base of reservoir: IV: 3.1H

f. Weir elevation: 676.5 feet

g. Normal Pool: 676.5 ± feet (east basin)
675-665 ± feet (west basin varies according to demand)

h. Storage capacity: 153 acre - feet

i. PMP 29.3 inches in 6 hours

j. ½PMP 14.6 inches in 6 hours

k. 100 year storm 4.8 inches in 6 hours

l. Size classification: Small

A.1.3 Cutoffs, Grout Curtains, and Liners: There are no known cut-offs or grout curtains installed other than the concrete that was placed in a few of the test pits that were excavated in 1920. In an effort to waterproof the reservoir, the interior walls were gunited in 1914 and 1921 and the floors were treated

with asphalt and covered with concrete. In 1977 the vinyl covers that protected the water surface of the reservoir were converted to liners in an effort to waterproof the structure.

A.1.4 Instrumentation and Monumentation: During the exploration program 10 inclinometers and 24 extensometers were installed to monitor movements of the foundation. In addition, water levels in some of the cored drill holes were also monitored. Data obtained from the inclinometers did not show any significant amounts of movement. The greatest amount did approach .5 inches, however most fell in the range of .01 to .02 inches which is within the error range of the instrument. The extensometers also did not show any appreaciable amount of movement. The monumentation that was installed consisted of 34 plugs set in the walkway on top of the reservoir. A Tennessee Highway Department brass marker is set in the walkway on the east side of the structure and all elevations of the plugs are referenced to it. Periodic surveys were made to detect any movement of the reservoir wall. No movement occurred.

Presently, these instruments and monuments are not monitored.

A.1.5 Operation and Maintenance: The structure and surrounding area is maintained by the Department of Water and Sewerage Services of Metropolitan Government of Nashville. The water level of the reservoir is maintained by the main water treatment plant located on Omohundro Drive in Nashville, Tennessee.

A.2 INLET AND OUTLET STRUCTURES

A.2.1 Inlet Pipe: The reservoir is filled by a 36-inch diameter pipe controlled by the main water treatment plant on Omohundro Drive.

A.2.2 Outlet Pipe: Water is distributed from the east basin of the reservoir by a 36 inch diameter pipe controlled by the Love Circle Reservoir.

A.2.3 Drawdown Facilities: The reservoir can be emptied through two,21 inch drains, one located in each basin. The valves to control these pipes are located in the gate house.

A.3 ADDITIONAL STRUCTURES

A.3.1 Gate House: The gate house is located on the north side of the reservoir. Three wells, influent, effluent and gate, are located in the gate house. The three wells house the various pipes and valves that regulate the water levels in the reservoir. The dimensions of the gate house are 27.3 by 55.7 feet.

A.3.2 Elevators, Stairs, and Storage room: Access to the gate house is

by either an elevator or set of stairs located on the north side of the structure. The elevator and stairs are part of the storage room constructed in 1977. Dimensions of this building are 41.3 by 25.6 feet.

A.3.3 Power Station: Located just to the north of the reservoir is the building which houses the pumps which distribute the water. The dimensions of this building are 16.5 x 19 feet.

A.4 HISTORICAL DATA

A.4.1 Original Construction Date: 1889

A.4.1.1 Failure and Reconstruction of East Basin Wall: 1912 to 1914
A.4.1.2 Underpinning of East Basin Wall: 1921
A.4.1.3 Ringwall and Tendon Installation: 1977

A.4.2 Designer: Original Structure - Unknown
1912 Reconstruction - Rudloph Hering
1920 Underpinning - Chester Engineers
1977 Tendon Installation - Chester Engineers and Geologic Associates

A.4.3 Builder: Original Structure - Whitsitt and Adams
1912 Reconstruction - Unknown
1920 Underpinning - Unknown
1977 Tendon Installation - Hardaway Construction Co.

A.4.4 Owner: Metropolitan Government of Nashville and Davidson County

A.4.5 Previous Inspections: None

A.4.6 Seismic Zone: 1

A.5 DOWNSTREAM HAZARD DATA

A.5.1 Downstream Hazard Classification: High

A.5.2 Persons in Likely Flood Path: Will vary depending on where failure of the reservoir would occur. It can range from 2 to 20.

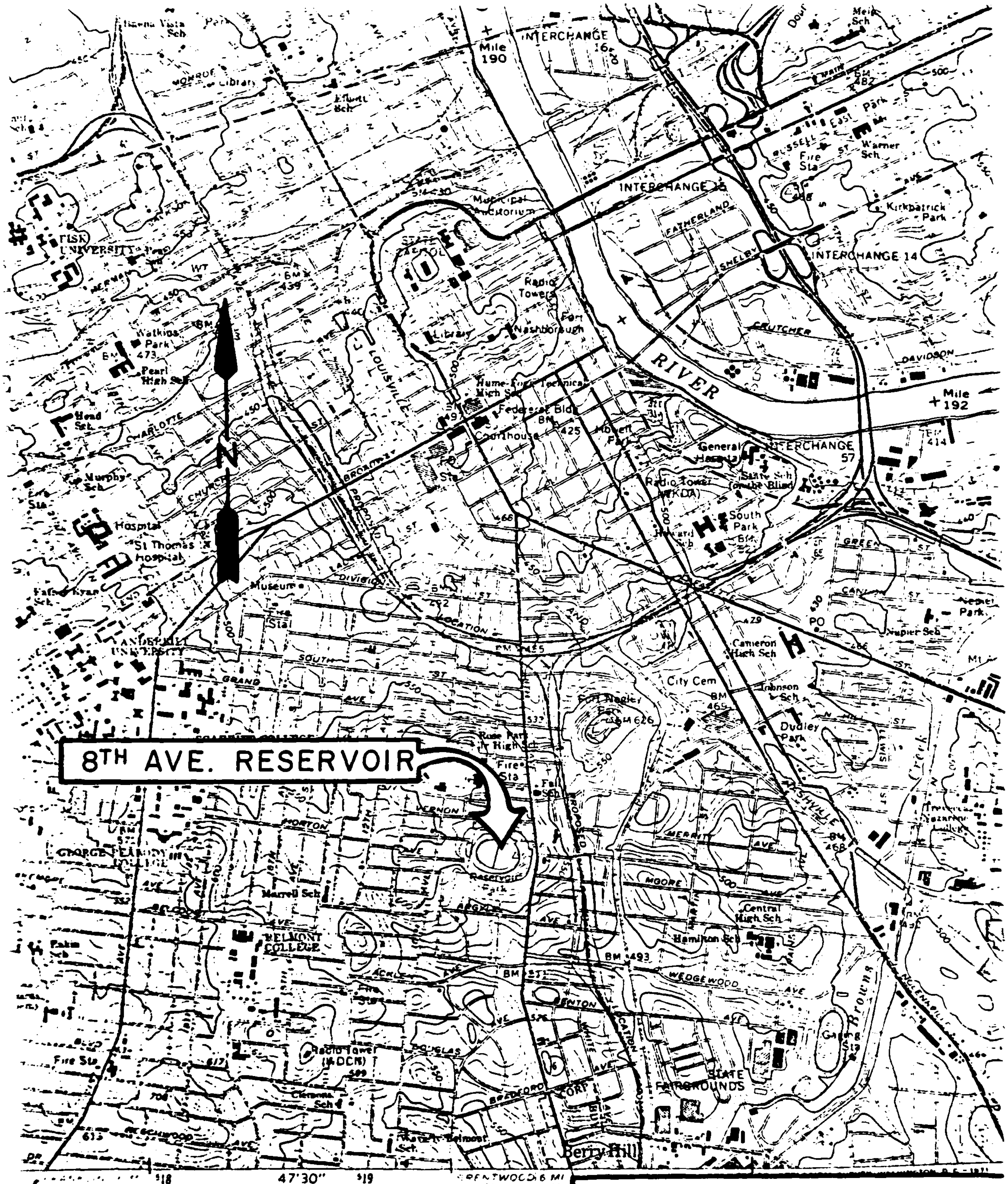
A.5.3 Downstream Property: East side of structure - 8th Avenue; South side - a park; West and North sides - apartment complexes.

A.5.4 Warning System: None

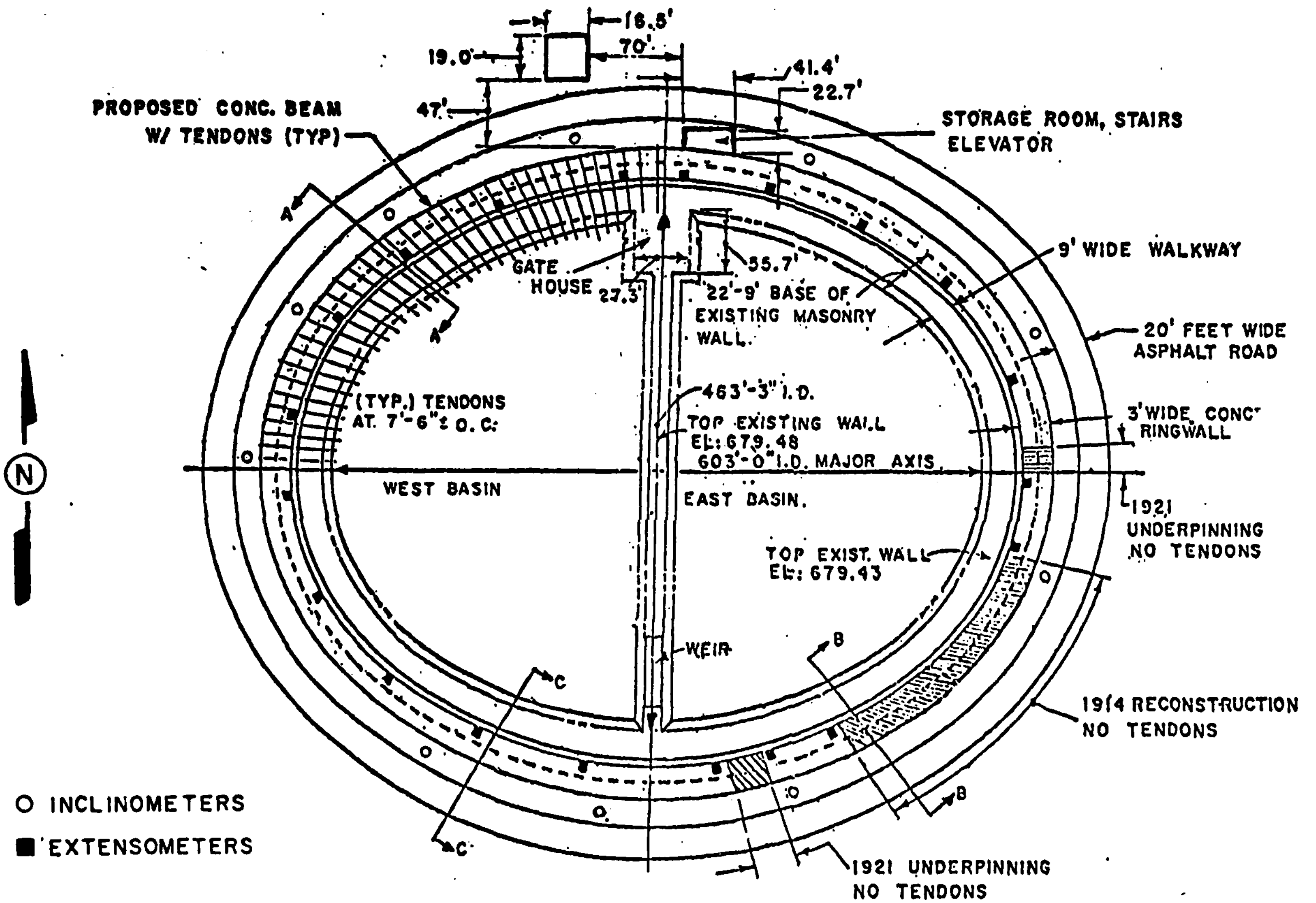
APPENDIX B

SKETCHES AND LOCATION MAPS

8TH AVE. RESERVOIR
NASHVILLE, TENNESSEE
LOCATION MAP
NASHVILLE
8TH AVE. RESERVOIR
OLD HICKORY DAM
J. PERCY PRIEST DAM
CORDELL HULL DAM
GALLATIN STEAM PLANT
CHEATHAM DAM
ISLANDS DAM
SEWART AIR FORCE BASE
ROBERTSON
SUMNER
SIMPSON
ALLEN
MACON
TROUSDALE
CHEATHAM
DAVIDSON
WILSON
WILLIAMSON
RUTHERFORD
CANNON
MAURY
BEDFORD
COFFEE
MARSHALL
CUMBERLAND
RIVER
DUCK
Springfield
Gallatin
Lebanon
Franklin
Murfreesboro
Columbia
Shelbyville
Lewisburg
Clarksville
Ashland City
Goodlettsville
Elkton
Scottsville
Portland
Woodbury
Manchester
Mt Pleasant

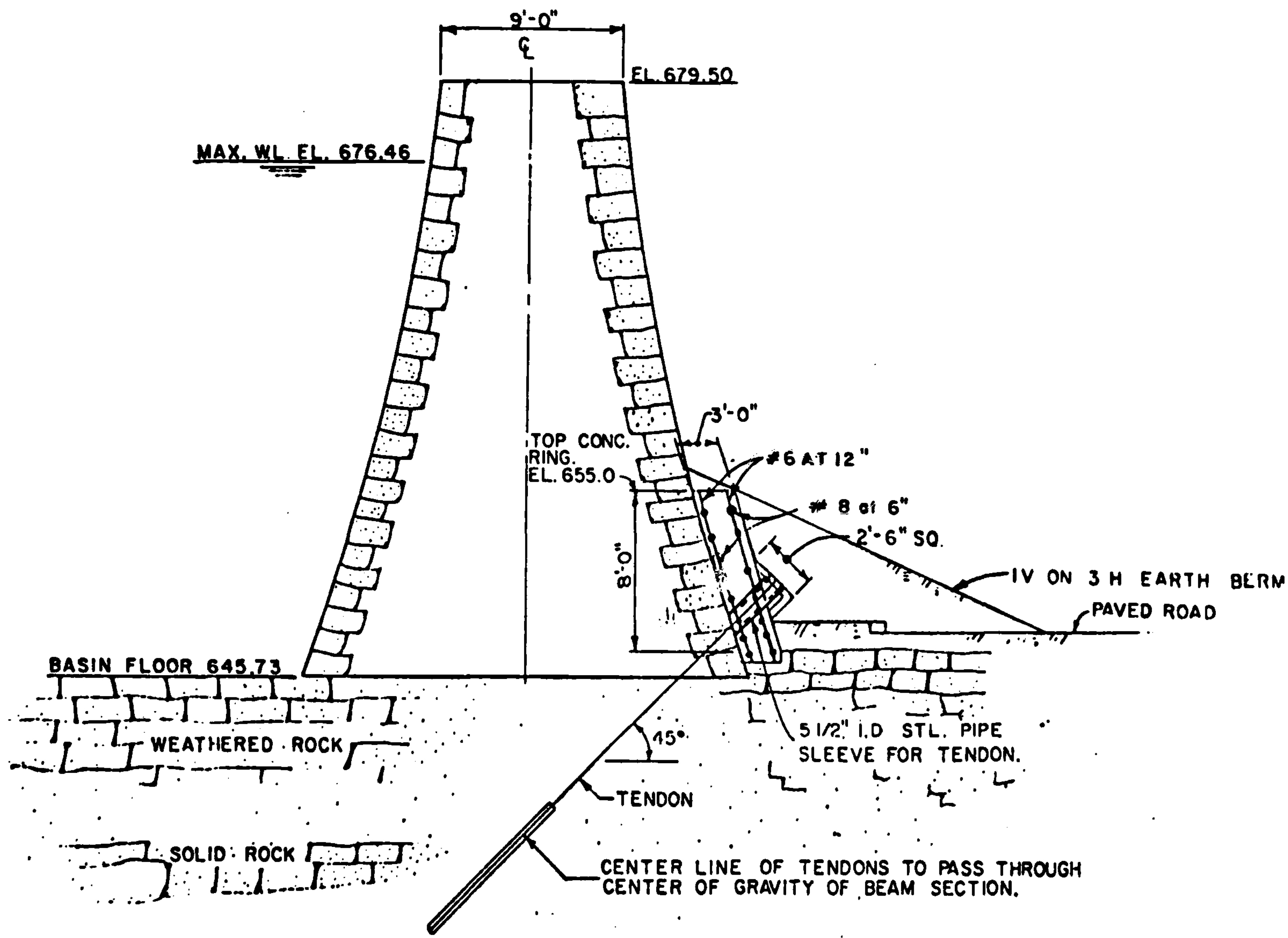


8TH AVE. RESERVOIR
NASHVILLE, TENNESSEE
SITE MAP



PLAN VIEW
NOT TO SCALE

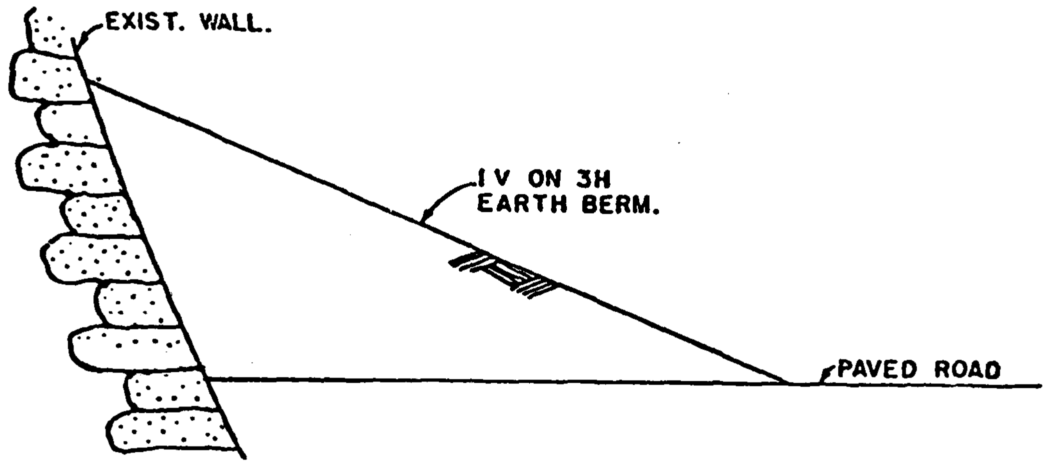
8th. AVENUE RESERVOIR
22 JUNE 1981
SHEET 1 OF 3



SECTION A-A

8th. AVE. RESERVOIR
NOT TO SCALE

8th. AVENUE RESERVOIR
22 JUNE 1981
SHEET 2 OF 3



SECTION B - B
8th. AVE. RESERVOIR
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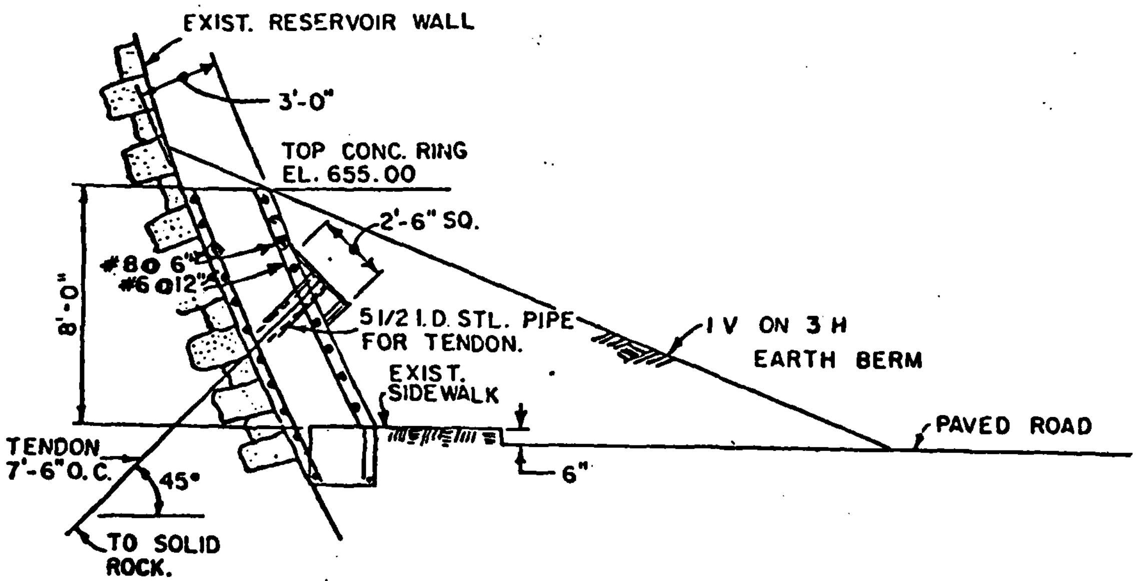


SECTION C - C
8th. AVE. RESERVOIR
NOT TO SCALE

8th. AVENUE RESERVOIR
22 JUNE 1981
SHEET 3 OF 3

APPENDIX C

PHOTOGRAPHIC RECORD

APPENDIX C
PHOTOGRAPHIC RECORD

Photograph No.	
1	Monument plaque in front of Reservoir
2	Top of Reservoir looking at gate house
3	Concrete spalling on walkway
4	Gate House Valves
5	Dry well of Gate House
6	Drain at bottom of dry well
7	Seepage in dry well
8	Section of wall rebuilt in 1914
9	Section of wall rebuilt in 1914
10	Calcium carbonate deposits on wall
11	Calcium carbonate deposits on wall
12	Deterioration of the stone
13	Exposed portion of concrete ringwall
14	Exposed portion of concrete ringwall
15	Inclinometer
16	Elevator, stairs, and storage building
17	Love Circle Pump House
19	Layout of Tendons
20	Drilling Tendon Holes
21	Reinforcing for concrete ringwall
22	Stressing the tendons

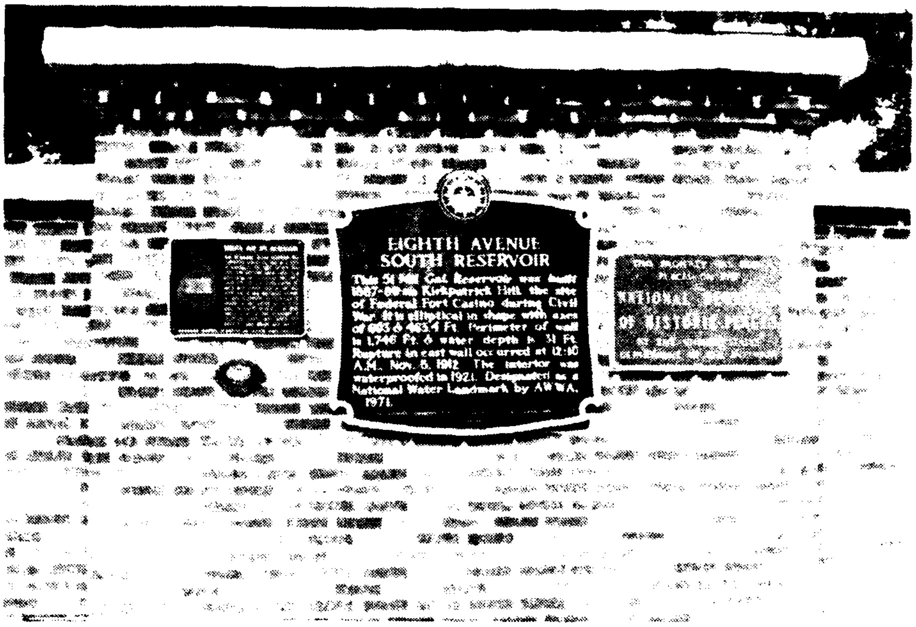


PHOTO NO. 1

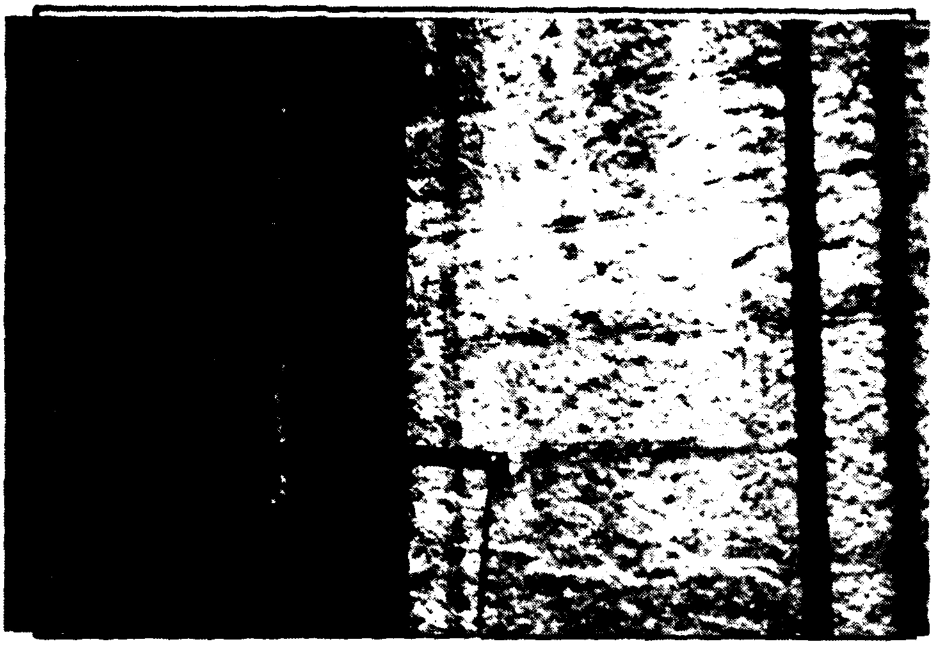
PHOTO NO. 2

PHOTO NO. 3

PHOTO NO. 4

PHOTO NO. 5

PHOTO NO. 6



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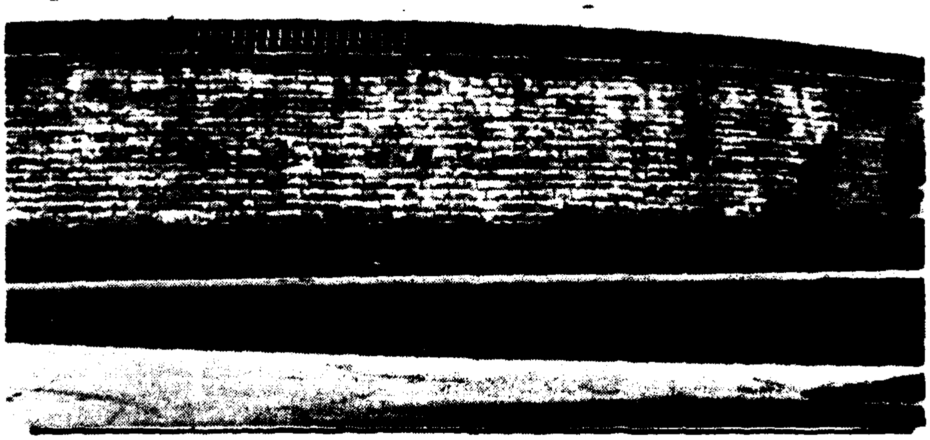


PHOTO NO. 8

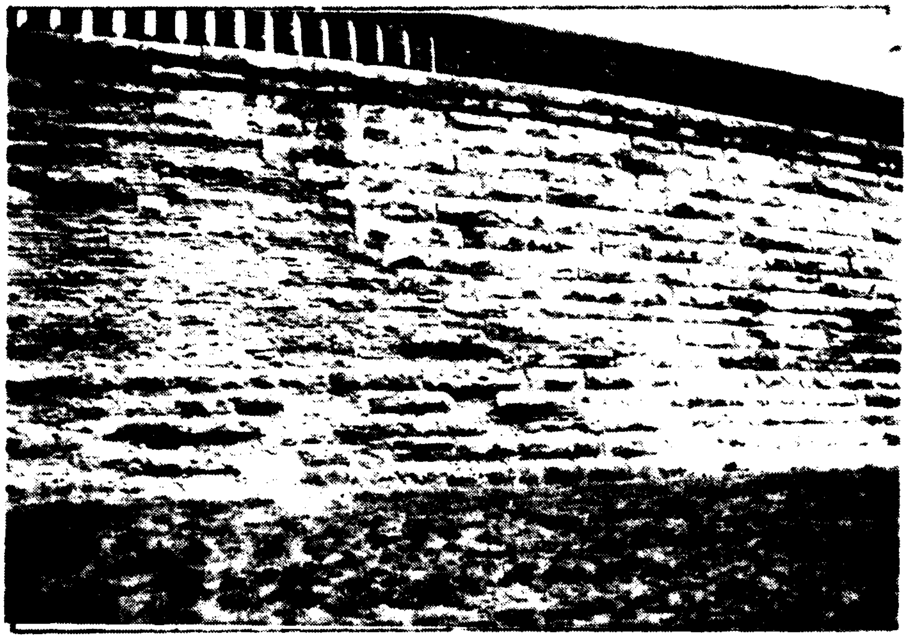


PHOTO NO. 9

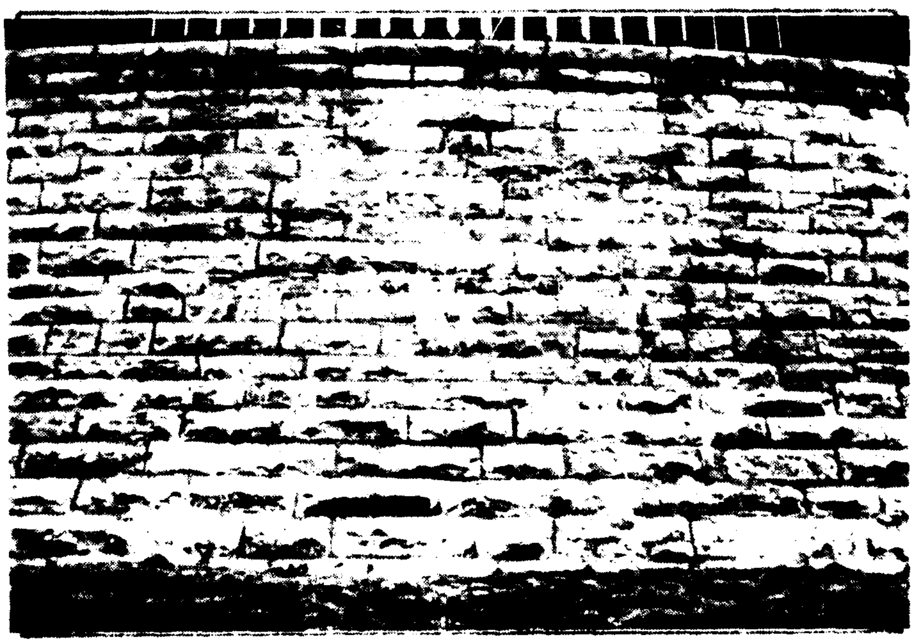


PHOTO NO. 10

PHOTO NO. 11

PHOTO NO. 12

PHOTO NO. 13

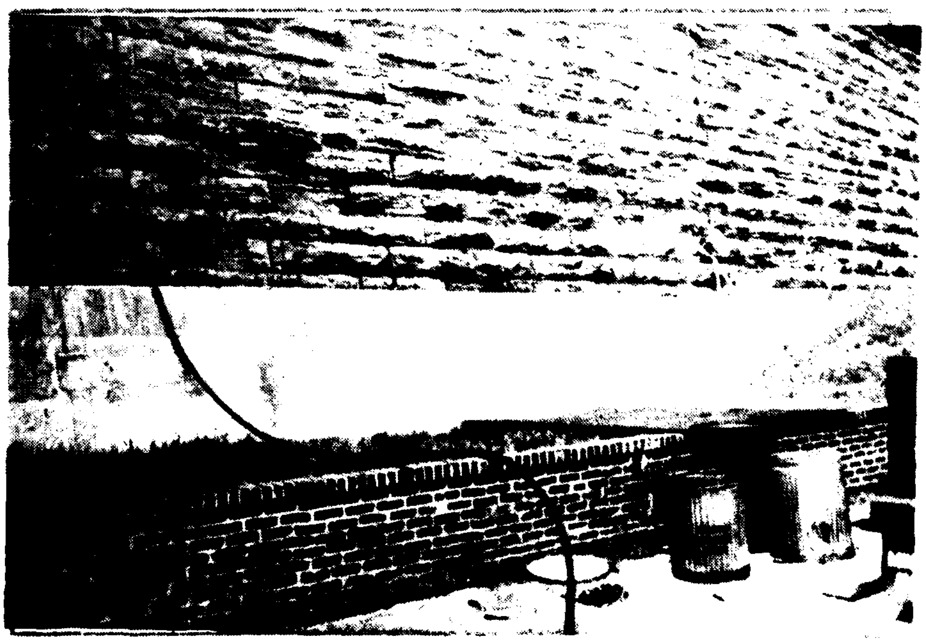


PHOTO NO. 14

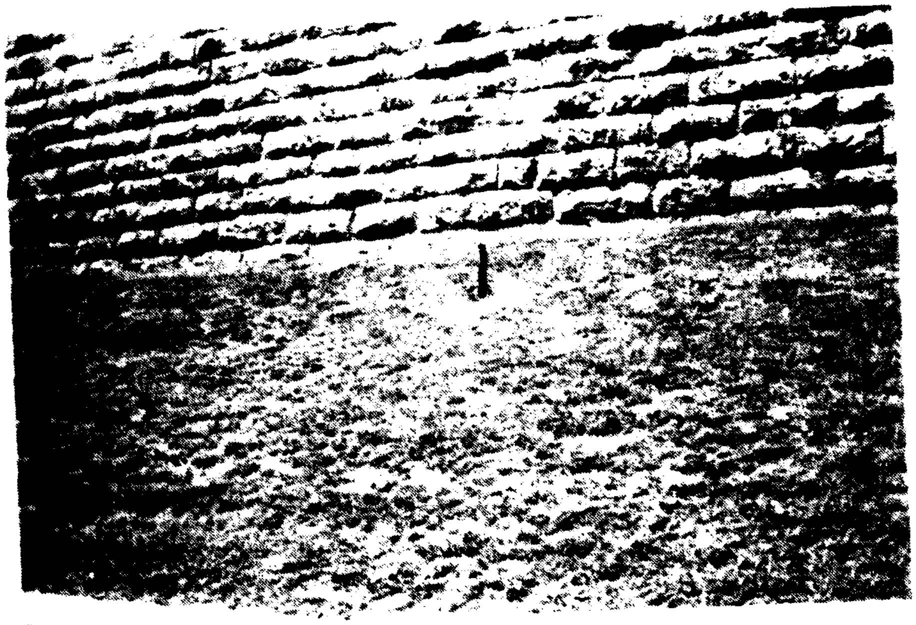


PHOTO NO. 15

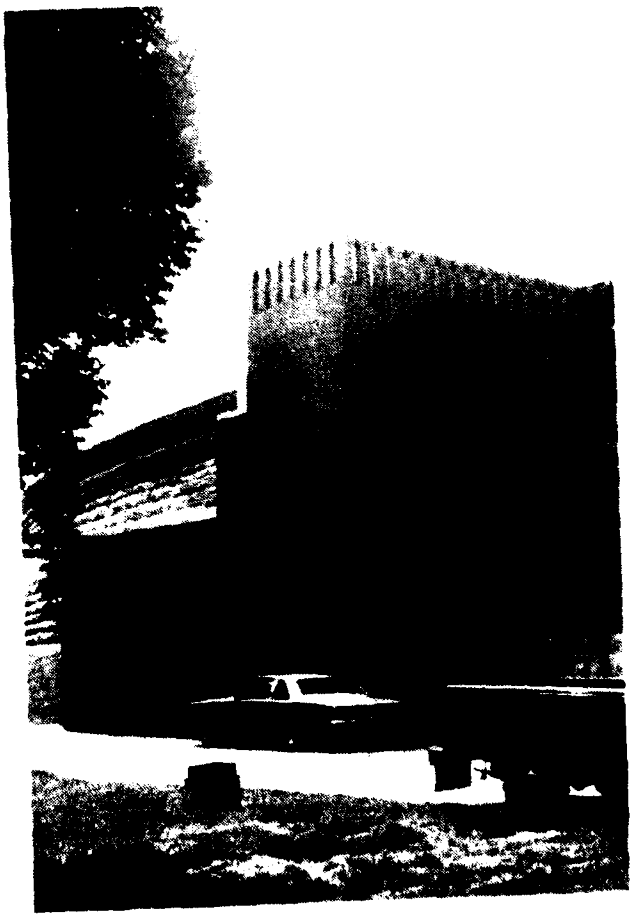


PHOTO NO. 16

PHOTO NO. 17

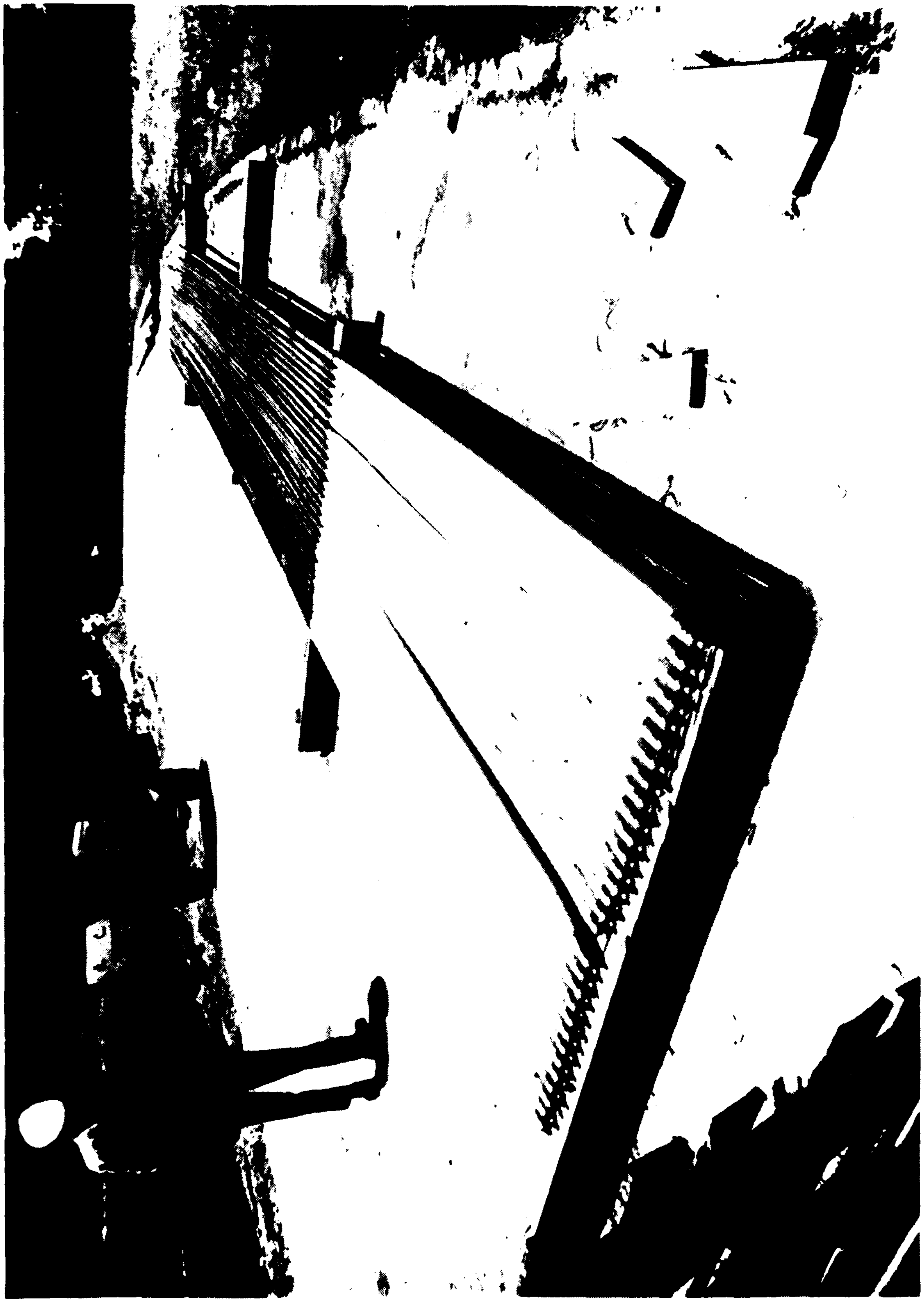
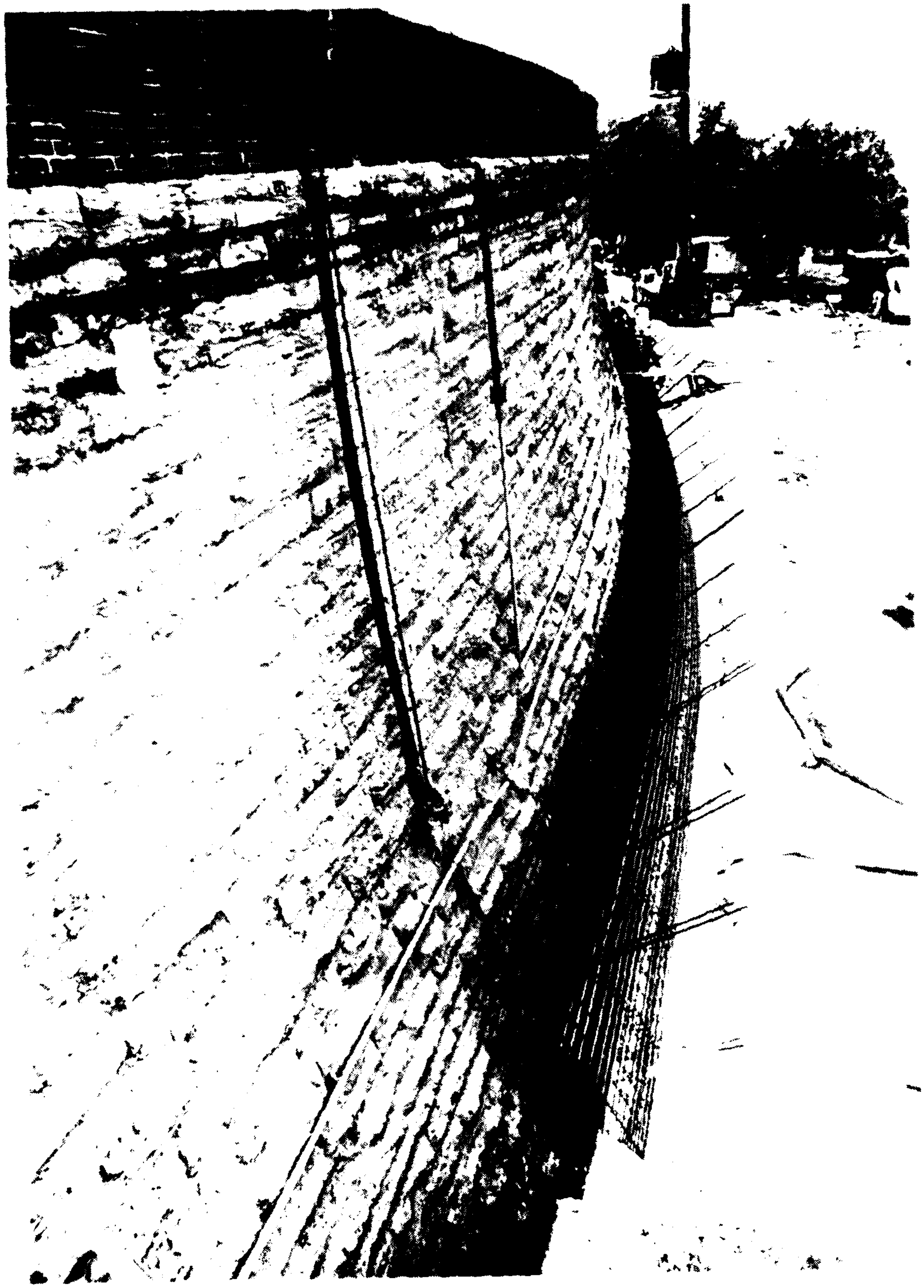


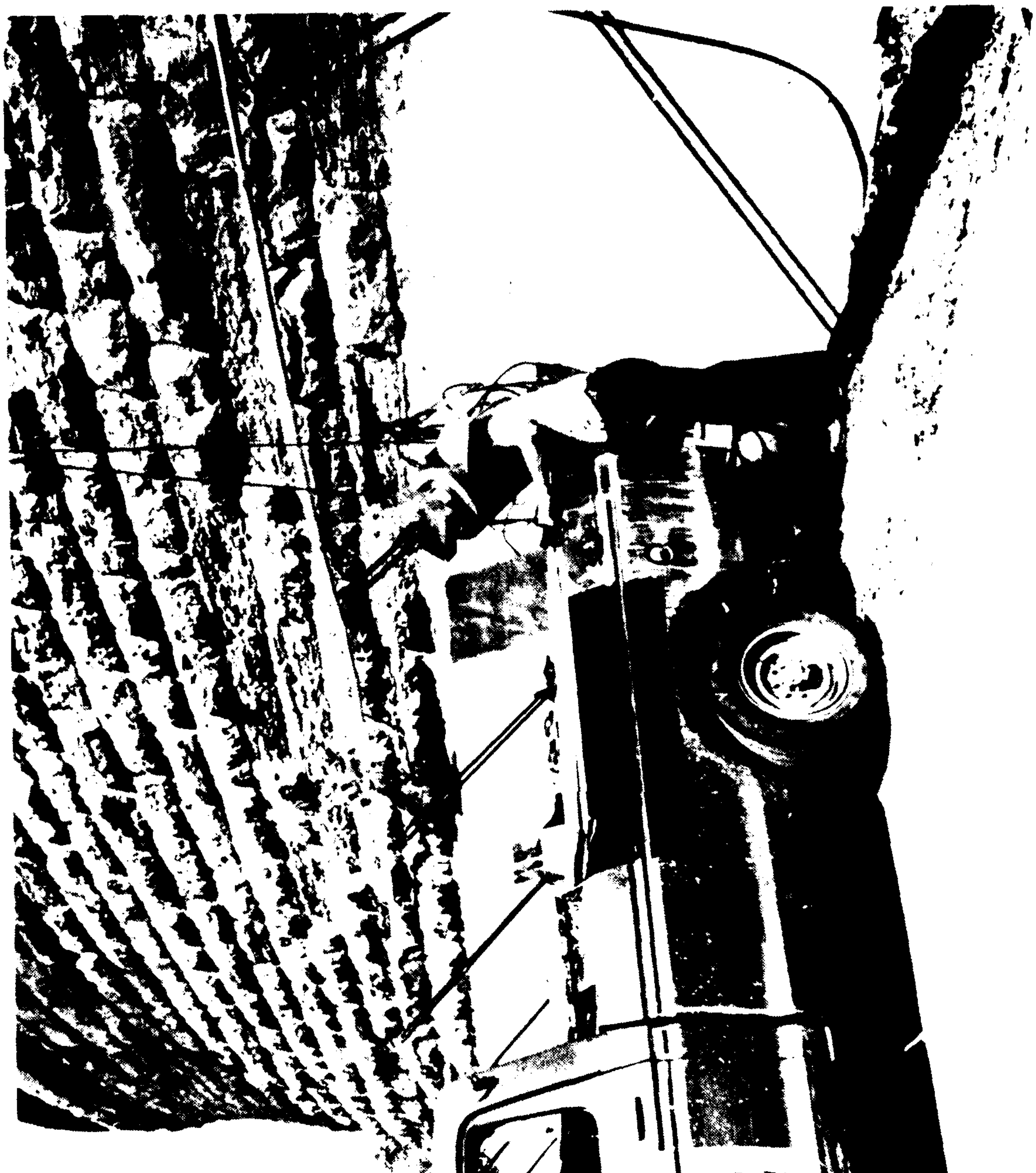
PHOTO NO. 19

PHOTO NO. 20



HARDAWAY CONSTRUCTION CO
Job #1179-07
7-15-77

PHOTO NO. 21



HARDAWAY CONSTRUCTION CO
Job #1179-07
9-21-77

PHOTO NO. 22

APPENDIX D

TECHNICAL CRITIQUES

ORNED-G

MEMORANDUM FOR RECORD

SUBJECT: Non-Federal Dam Inspection of 8th Avenue Reservoir

1. As part of the Non-Federal Dam Inspection Program, the 8th Avenue Reservoir was inspected on 29 June 1981. Below is a list of personnel who took part in the inspection.

Nashville District - Corps of Engineers

Paul Bluhm	Inspection Coordinator
Tom Porter	Hydraulic Engineer
Tom Allen	Hydraulic Engineer
Gordon McClellan	Structural Engineer
Randy Bush	Soils Engineer
Wayne Swartz	Geologist

Tennessee Department of Conservation

Bill Culbert	Water Resources Engineer

Metropolitan Government - Department of Water and Sewarage Services

Gene Johnson
Fred Clinare

2. Mr. Gene Johnson first gave a brief summary of the history of the reservoir. The reservoir was constructed in 1889 to be used as a water supply for the city of Nashville. It is elliptical in shape and divided into two basins. Originally water was pumped from the Cumberland River into one basin where it was allowed to settle and then pumped to the other basin for distribution. A section of the east wall of the structure failed in 1912, but was repaired by 1914. In 1920, portions of the wall near the 1912 break were underpinned. No additional remedial work was performed on the structure until 1975 when leaks were observed on the north side of the reservoir. An extensive exploration program was undertaken to determine the cause or source of leakage. The end result of this study was the installation of a tendon through the reservoir which was anchored in rock. A reinforced concrete ring wall, 8 feet in height was also constructed around the reservoir (except in the area of the 1914 and 1920 work) covering the tendons. In addition to this work, an elevator and stairs were also added to the structure. No other remedial work has been performed.

3. An inspection was then made on the reservoir. There were three main areas to be inspected: The gate house, the top and interior of the reservoir.

and the exterior and surrounding area of the structure.

a. Gate House - The control room is located on the north side of the structure and houses the valves which control the water levels in the two basins. The structure is divided into three sections or wells; the influent, effluent and gate wells. Water is pumped in to the influent well from the main pumping station on Omohundro Drive through a 36 inch pipe. The east basin is then regulated by two 36 inch pipes from this well while the west basin is regulated by a weir located in the wall separating the two basins Distribution is made from the west basin through two 36 inch pipes located in the effluent well. Distribution can also be made from the east basin. but the valves on the two 36 inch pipes are normally closed. The valves for the 21 inch drains of both basins are located in the gate well as are the valves for the drains of the other two wells. The interior of the gate well was in good condition although some leakage was observed. The seepage did not appear to be serious and drains at the bottom of the gate well removed this leakage. The pipes and valves appeared to be in good condition although the valves were not operated. Control of the water level in the reservoir by the main pumping station at Omohundro Drive.

b. Reservoir - top and interior: The top of the reservoir wall provides a walkway around the entire structure. It is concrete, 9 feet in width and has a 4 foot high brick parapet wall on its outer edge and a railing on the inner edge to provide protection. The center wall, dividing the two basins, also provides a walkway and railing. A weir, which regulates the water elevation in the two basins, is located at the south end of the center wall. The walkway covers the weir and it was not observed at the time of the inspection. The reservoir itself was covered with a strong, vinyl cover which was required of the owner by law. There the interrior walls could not be inspected. The water elevation at the time of inspection was approximately 3 feet below the top of the wall. A Tennessee Highway Department brass marker was set in the wallway on the east side and is used to determine the elevation of 23 plugs that were set at various points around the top of the reservoir wall. These plugs were installed to determine if the wall settled during the recent remedial work. Overall, the top and interior of the reservoir was in good condition with the exception of some minor concrete spalling which is occurring on the walkway near the gate house.

c. Reservoir: Exterior - The reservoir wall has according to plans, a base width of 22 feet 9 inches and a height of 34.5 feet. A concrete ring wall, constructed in 1978 when the tendons were installed, encircles all of the reservoir with the exception of the 1914 and 1921 reconstruction. The ring wall is 8 feet in height and is visible only in the area near the stairwell for the gate house. A IV on 3H earth berm extends from the top of the concrete wall to the asphalt road and encircles the entire reservoir leaving 26 feet of the reservoir wall visible for inspection. There are 8 inclinometers and 20 extensometers located at various points around the reservoir (on the earth berm). The inclinometers were protected by steel casings that extended about 1.5 feet above the ground surface. The conduits

for the extensometer extended through the wall and were routed to the control room where they can be read electronically.

The stairwell for the gate house is located on the north side of the structure as is the elevator, guard room and chlorine cylinder storage. Located directly across from these buildings is a pump house and guard house. No other structures are located near the reservoir.

The conditions of the reservoir appeared to be very good. Portions of the wall showed signs of some deterioration but it did not appear to be serious. Some areas of the original wall and the 1914 reconstructed wall showed signs of leakage as evidenced by calcium deposits. Leakage was observed but it was very minute and generally not noticeable. Overall the masonry wall appeared to be in good condition. The area and foundation immediately surrounding the reservoir did not show any wetspots, areas of seepage or any signs of distress. Maintenance of the area immediately adjacent to and surrounding the reservoir was very good. The grounds were mowed and well kept.

5. Conclusions and Recommendations

a. Structurally the reservoir appeared to be in very good condition. There were no signs of cracks, differential settlement, or wet areas on or near the reservoir.

b. The slight leakage that was observed was not considered serious.

c. The spalling of the concrete that was observed on the walkway was not serious, but should be repaired.

d. The valves and pipes in the gate house appeared to be in good condition.

e. The leakage that was observed in the gate well was not considered serious.

f. Inclinometer caps should be provided for all inclinometers.

g. The owner should establish a regular schedule for reading and reviewing the data from the inclinometers and extensometers.

h. An emergency action plan should be developed to warn downstream residents in the event a serious problem develops with the structure.

Paul F. Bluhm
Civil Engineer

ORNED-G

MEMORANDUM FOR RECORD

SUBJECT: Inspection of Metropolitan Nashville 8th Avenue Reservoir

1. Purpose: To inspect and render judgment as to the adequacy and acceptability of the safety of the 8th Avenue Reservoir under the Safety of Non-Federal Dams program.

2. Place and Date: Metropolitan Nashville 8th Avenue Reservoir, 29 June 1981.

3. Persons Making Inspection:

Tommy Allen	ORNED-H
Paul Bluhm	ORNED-G
Randy Bush	ORNED-G
Gordon McClellan	ORNED-D
Tom Porter	ORNED-H
F. Wayne Swartz	ORNED-G
Bill Culbert	Tenn. Dept. of Water Resources

4. Persons Contacted:

Fred Clinard	Metro Dept. of Water and Sewerage Service
Gene Johnson	Metro Dept. of Water and Sewerage Service

Background: The 8th Avenue Reservoir is an elliptical shaped gravity-type wall structure constructed of cut stone masonry facing with "cyclopean concrete" between facings. The major axis is 603 feet in length and the minor axis is 463.4 feet in length. A wall along the minor axis divides the reservoir into two basins of more than 25,000,000 gallons capacity each. The top of the wall is at approximate elevation 676 ± with the basin floor at approximate elevation 643 ±.

The structure, constructed in 1887-89, is located on the west flank of the Nashville Dome. There is a gentle NW areal dip of beds with many variations and reversals of the regional dip. The reservoir is constructed on the shaly limestones of the Ordovician age Catheys formation which dips slightly W-NW and is weathered to depths of 10-25 feet. Based on studies conducted by Geologic Associates Inc. in 1975, the reservoir foundation is crossed by three roughly parallel gravity faults.

A failure of the southeast section of the wall resulted in extensive repairs which were completed in 1914. Signs of structural distress necessitated additional remedial treatment bracketing this area in 1921. The 1912 failure and past leakage problems have been attributed to founding the structure on the highly weathered limestone. Remedial

foundation treatment to provide an acceptable factor of safety was performed in 1977.

Prior to the inspection, I reviewed the reports covering the sub-surface investigations, the analysis and evaluation of the data, and the stabilization that was provided. The entire program seems to have been very well handled. The investigations and the stability analysis were very thorough in addressing the problem and its solution. The remedial measures that were taken should be adequate to provide the factor of safety that would necessarily be required.

6. Observations: Following a briefing by Messrs. Clinard and Johnson, the group separated to inspect the structure. The overall condition of the structure appears acceptable with no evidence of foundation distress. However, the following deficiencies were noted:

a. Although there was no seepage at the foundation or base of the wall and overall no significant seepage, there was minor seepage in association with calcium carbonate deposits, especially on the SE wall.

b. There has been some deterioration of the outer stone facing and mortar which may be attributed in large part to the poor weathering resistance of the shaly limestone used in the reservoir's construction. It may be that increased air pollution has and will accelerate this deterioration, but it is not a serious problem at this time.

c. A number of inclinometers were found without protective covers.

7. Conclusions and Recommendations: Based on a review of available geotechnical data and on the on-site inspection, the 8th Avenue Reservoir is in good shape with no signs of foundation distress. The minor seepage as well as the deterioration of the rock facing should be closely monitored for changes, especially as to rate and/or quantity.

F. Wayne Swartz

F. WAYNE SWARTZ
Geologist
Geotechnical Branch

CF:

ORNED-G (Bluhm)

SIMMONS ED-G
COUCH ED-G
EASTLAND ED
MOORE ED

ORNED-D 7 August 1981

MEMORANDUM FOR RECORD

SUBJECT: Trip Report - 8th Avenue Reservoir Inspection

1. On 29 June 1981 I accompanied the inspection party for the inspection of the 8th Avenue Reservoir, Nashville, Tennessee under the Non-Federal Dam Inspection Program. The Reservoir is elliptical in plan with a major axis of 603 feet and a minor axis of 463.4 feet. The reservoir is split along the minor axis with a divider wall. The tops of the walls are 8 feet thick and have a thickness of approximately 23 feet at the floor of the reservoir. The walls are faced with cut stone masonry and are filled with stone rubble. Each basin of the reservoir holds approximately 25,000,000 gallons and they are lined and have a floating cover. There is a tensioning ring for the prestressed anchors at the foot of the wall.

2. The facing stone masonry on the exterior of the reservoir has weathered and the thickness of the stone has been reduced. The original stone has weathered much worse than the stone used in the 1912 repair. There is very little seepage through the wall and the wall was dry except in one place in the rebuilt section. The addition of the liner in 1976 has probably slowed the weathering of the stone, however, this should be monitored.

3. The tensioning ring is visible only in the area of the pump house itself, since it has been covered with an earth berm. The top edge of the beam shows through at several locations. The concrete of the beam is in good condition and there is no rust showing in the joints for the grout caps over the anchorages for the prestressing tendons.

4. The concrete on top of the wall is cracked and spalling. We were told that the wall would be capped with asphalt.

Gordon McClellan

GORDON MCCLELLAN
Civil Engineer

Project History

The Eighth Avenue Reservoir is located on Kirkpatrick Hill and is owned by the Metropolitan Government of Nashville and Davidson County. Construction was begun in August 1887 and the reservoir was finished in 1889. The reservoir is elliptical in plan with a major axis of 603 feet and a minor axis of 463.4 feet. There is a dividing wall along the minor axis that divides the reservoir into two basins of approxiamtely 25,000,000 gallons. The walls are 8 feet thick at the top and approximately 23 feet thick at the floor of the reservoir. At some locations, it has been determined that the wall is 36 feet or more at its contact with the subgrade. The walls are faced with cut stone masonry and filled with stone rubble. The interstices of the rubble were filled with concrete to produce a mass referred to as "cyclopean concrete." Originally, the dividing wall was used to separate the settling basin from the treated water basin. The two basins are now connected, with the treated water entering the east basin and exiting from the west basin.

On November 5, 1912, a segment of the east basin wall, approximately 200 feet in length, was displaced and that basin emptied. Old photographs have shown that the force of the flow was sufficient to sweep away the fill placed on the hillside and scour the natural ground to bedrock. A segment of about 100 feet in length remained intact as it was displaced.

The wall was rebuilt in 1914. In addition to rebuilding the wall, a perimeter french drain system was installed adjacent to and inside the east basin wall. At the same time, the interior walls were gunited and the floor of the basin treated with asphalt and covered with concrete. Six test pits were dug around the west basin and one buttress was built.

In 1920, the east basin was withdrawn from service after an inspection revealed a horizontal crack in the masonry wall. This was in the segment adjacent to and north of the section that failed in 1912. A study was implemented which included the use of core holes and test pits, and lead to the underpinning of about two-thirds of the affected area in 1921. As part of this work, the walls of the reservoir were scaled and were coated with a new layer of gunite, and a new concrete floor was installed.

In 1974, contracts were let to install covers over the two open basins. The contracts included cleaning the reservoirs, guniting the interior walls, and installing butyl rubber floating covers. This work was completed in March 1975.

After the reservoir had been returned to operation in the spring of 1975, seepage was noted in the lower courses of masonry along the northwest arc of the west basin. This seepage was greater than had been observed previously, and it was decided to drain the west basin. A subsurface investigation was begun, and from the data obtained, it was determined that the subsurface was inadequate to provide a sufficient factor of safety against sliding. A tensioning ring with tendons anchored in sound rock was installed around the reservoir, except in the areas of the 1912 and 1920 repair work. At the same time, the floating covers were converted to liners for the basins and new covers installed.

Review of Repairs

The failure of the reservoir in 1912 seemed to be related to the foundation material rather than the structural stability of the walls. Geologic Associates concluded that the failure was caused by piping of the subgrade. They based their findings on photographs of the 1912 failure and the fact that an almost 100 foot segment of the wall remained intact after being displaced. They felt that the failure was a typical wedge failure, with the wall being displaced downward and outward.

The failed portion of the wall was rebuilt with a better grade stone than the original stone. It is not as susceptible to weathering, and it is very apparent where this new stone was used, since it is a different color.

The 1921 repair work consisted of excavating narrow trenches under the wall to sound rock and backfilling with concrete. This extended the walls to sound rock. This work was considered to be adequate when the repairs were made in 1975.

The 1975 repair work consisted of adding a tensioning ring with tendons anchored in sound rock. This system was designed by the Chester Engineers. The ring was installed around the reservoir, except in the areas of the 1914 and 1920 repair work. Tendons were then installed through this ring. The tendons consisted of Dywidag Threadbars on 7'6" centers. The bars were designed for a load of 160 kips, but were load tested to 239 kips. Each bar was embedded a minimum of 10'0" into sound bedrock This medthod raised the factor of safety against sliding to about 1.5. This method was selected after reviewing several alternatives, including abandonment of the present reservoir and building on a new site.

ORNED-G 9 July 1981

MEMORANDUM FOR RECORD

SUBJECT: Trip Report - Inspection of Metro Nashville 8th Avenue Reservoir

1. The Metro Nashville 8th Avenue Reservoir was inspected on 29 June 1981 in conjunction with the Non-Federal Safe Dams Program. At the site we were briefed on the structure history by Metro water-sewer personnel. The structure was completed in approximately 1889, rebuilt after a failure in 1914 and the walls outside the failure area renovated by underpinning in approximately 1921. The underpinning consisted of removing the shaley rock beneath the footing and replacing it with concrete down to sound rock. A concrete (reinforced) collar was constructed around the base of the structure, exclusive of where the structure was repaired and tendons installed through the collar into sound bedrock a few years ago. In recent years a plastic liner and cover has been installed on the reservoir. The walls of the reservoir are approximately 26 feet thick at the base. According to water-sewer personnel the walls contain a clay core, however, some reports say the walls contain cyclopian concrete so it is uncertain as to what the interior of the walls contain.

2. A very slow dripping of water from a stalagtite formed on the wall approximately 8-10 feet up from the ground was observed. This was within an area of the wall which showed mineralization and within the repaired section of the wall. Other areas of the wall showed signs of mineralization occurring (wall turning white) and stalagtites forming which indicates possible seepage through the wall in the past.

3. Some of the stones of which the walls are built are scaling. Some of the stones are scaling noticeably faster than others, particularly a section of the wall near the repaired section. At this time, however, the scaling of the rock appears to present no immediate problems.

4. No seepage or soft spots were observed at the base or around the structure area. The structure appears stable and in good condition. Five inclinometers at the base of the structure did not have caps and two of these instruments were full of water. Caps should be provided for these instruments to protect them.

Randy H. Bush

RANDY H. BUSH
Soils-Embankment Design Section
Geotechnical Branch

DISPOSITION FORM

For use of this form, see AR 340-15; the proponent agency is The Adjutant General's Office.

REFERENCE OR OFFICE SYMBOL	SUBJECT
ORNED-H	Inspection of the 8th Avenue Reservoir at Nashville, Tennessee for the Safe Dam Inspection Program on 29 June 1981.

TO Chief, I&I Branch **FROM** Chief, H&H Branch **DATE** 19 August 1981 **CMT 1**

JNP Porter/ /5632

1. The 8th Avenue Reservoir is fed by the distribution network of Metro. The reservoir is divided into two compartments. Each compartment can hold 25 million gallons. Water flows from one side to the other by gravity. Both compartments can be drained independently of the other.

2. Water can be drawn out by either emptying into the water distribution network or by dumping into the sewer system.

3. The reservoir is covered by a tarp which eliminates debris/bird droppings from the water.

4. Hydrologic failure is not being considered because the reservoir is completely enclosed and has a controlled supply and drainage system.

5. In the event of structural failure the path of flooding would be dependent on the path of structural failure. ~~can not be predetermined.~~ The ground surrounding the reservoir is reasonably level but drops off rapidly on all four sides about 100 feet away from the structure to apartments and commerical buildings. Should the reservoir fail it seems reasonable that the potential

for the loss of life is very great. Apartments are located within several hundred feet on two sides with Reservoir Park on the third side and commercial buildings on the fourth (8th Avenue).

6. A review of the piping arrangement has been made from the blueprints and during the site inspection. The piping observed appears to be adequate.

Williams DW

Phillips P

Connor

Allen A

APPENDIX E

DESIGN DRAWINGS

2

METROPOLITAN GOVERNMENT OF NASHVILLE AND DAVIDSON COUNTY, TENNESSEE

DEPARTMENT OF WATER AND SEWERAGE SERVICES

8TH AVENUE SOUTH RESERVOIR

PROJECT	NO. 76-WC-47A	STRUCTURAL STABILIZATION IMPROVEMENTS
PROJECT	NO. 76-WC-47B	MISCELLANEOUS IMPROVEMENTS
PROJECT	NO. 76-WC-47C	LINING & COVER

RICHARD FULTON

METROPOLITAN MAYOR

K. R. HARRINGTON

DIRECTOR OF WATER AND SEWERAGE SERVICES

SUBMITTED BY THE CHESTER ENGINEERS

BY ______________ TENNESSEE CERTIFICATE NO. 426   11-10-76 DATE

RECOMMENDED FOR APPROVAL

BY ______________ ASSISTANT DIRECTOR OF WATER AND SEWERAGE SERVICES   11/10/76 DATE

APPROVAL BY ______________ DIRECTOR OF WATER AND SEWERAGE SERVICES   DATE

APPROVED BY ______________ MAYOR   DATE

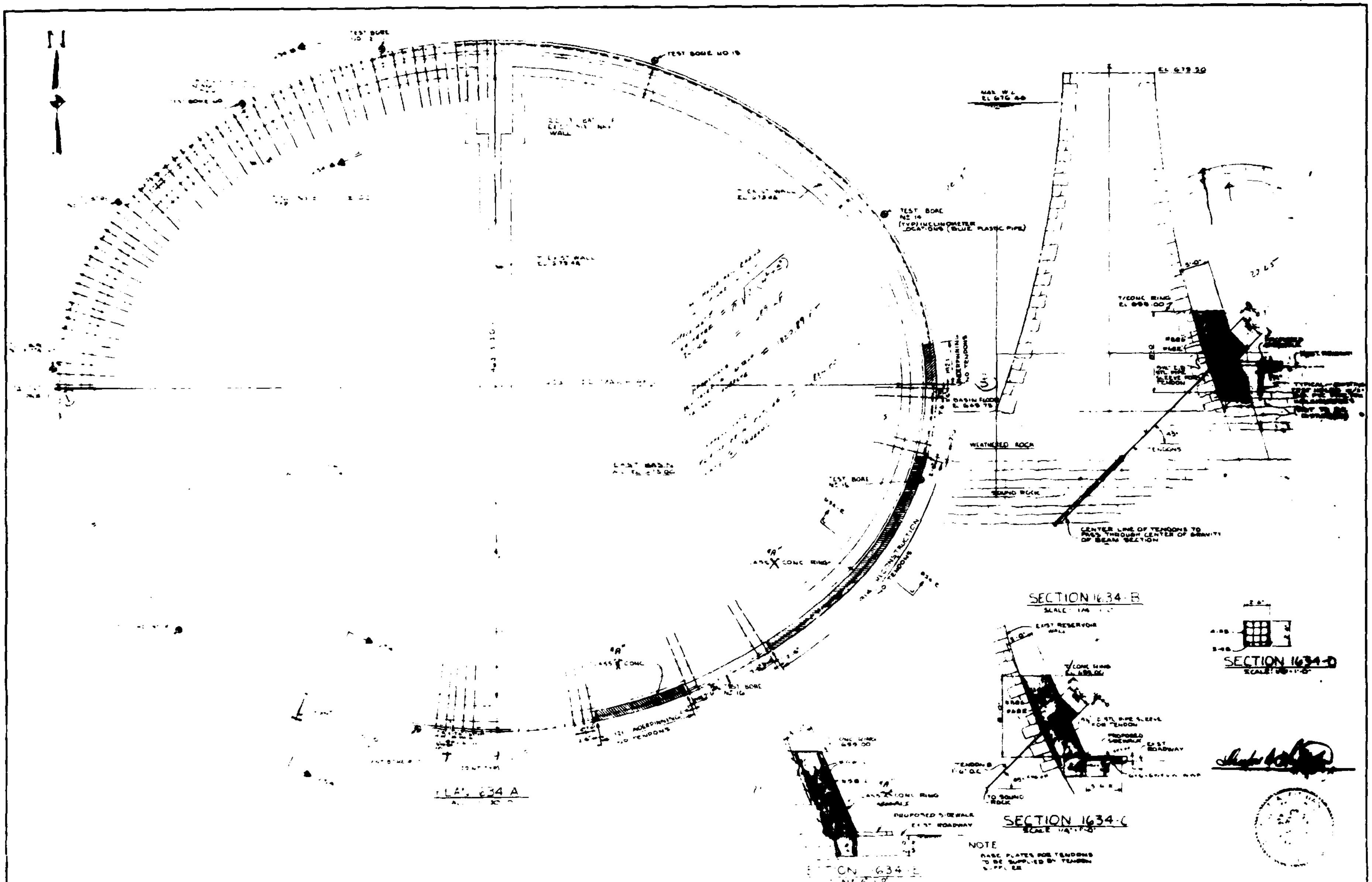
REVISED DECEMBER, 1976

OCTOBER, 1976

THE CHESTER ENGINEERS

CORAOPOLIS, PENNSYLVANIA

ATLANTA, GEORGIA



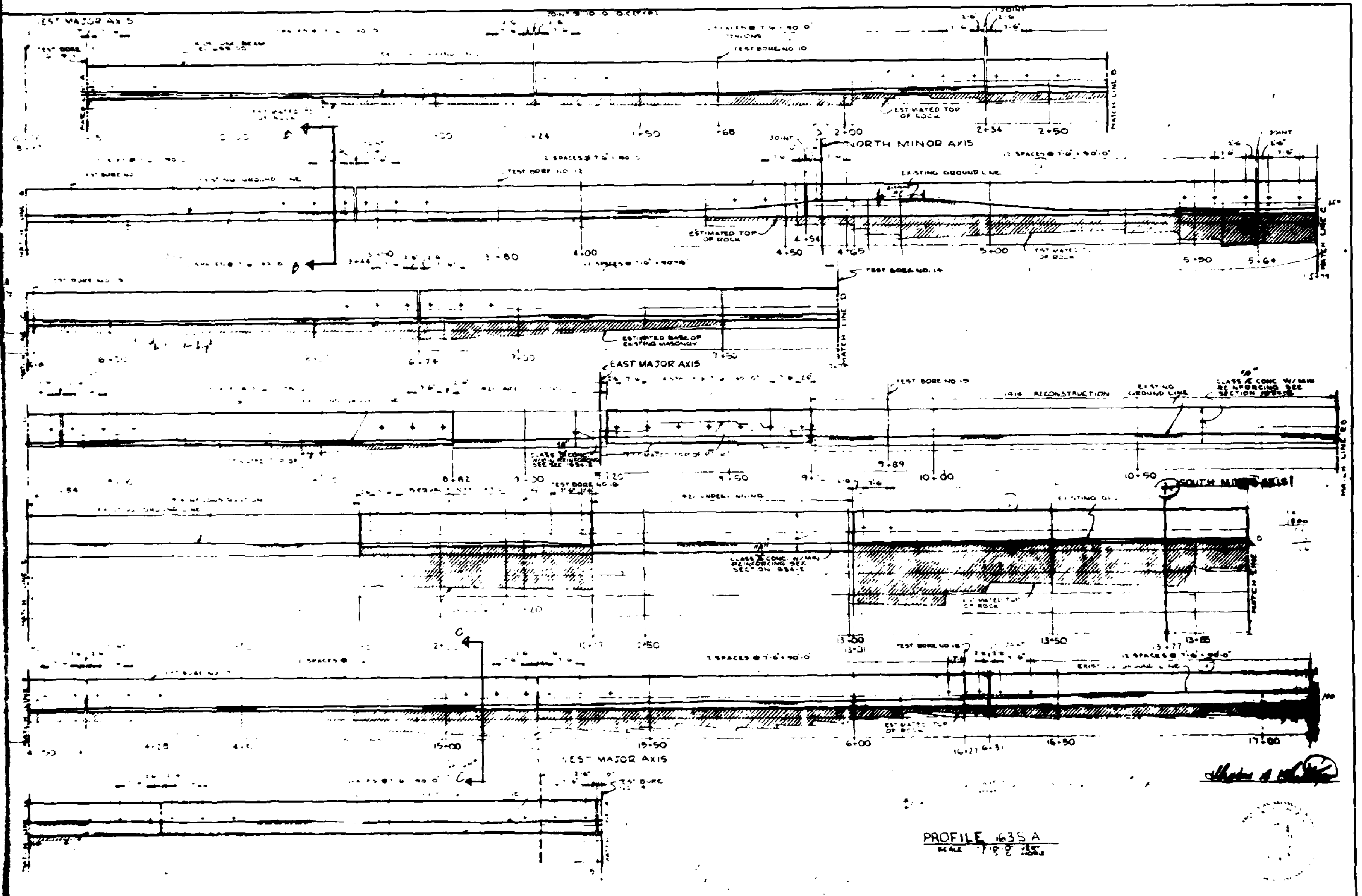
SCALE
DATE

THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

METROPOLITAN GOVERNMENT OF NASHVILLE AND DAVIDSON COUNTY, TENNESSEE
DEPARTMENT OF WATER AND SEWERAGE SERVICES

EIGHTH AVENUE SOUTH RESERVOIR
PLAN AND SECTION

DWG. NO. 000-1634
SHEET NO. 1 OF 12

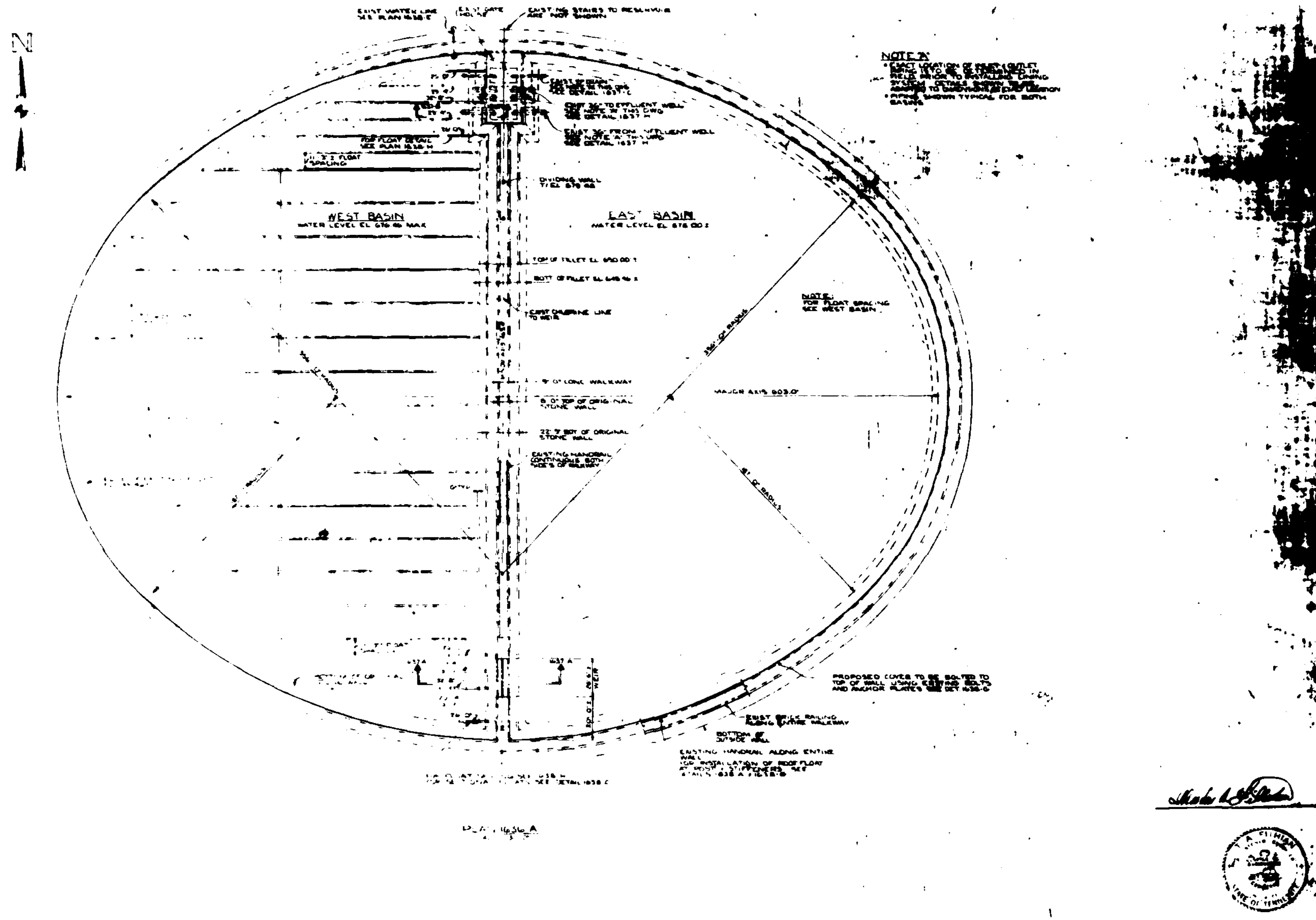


THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

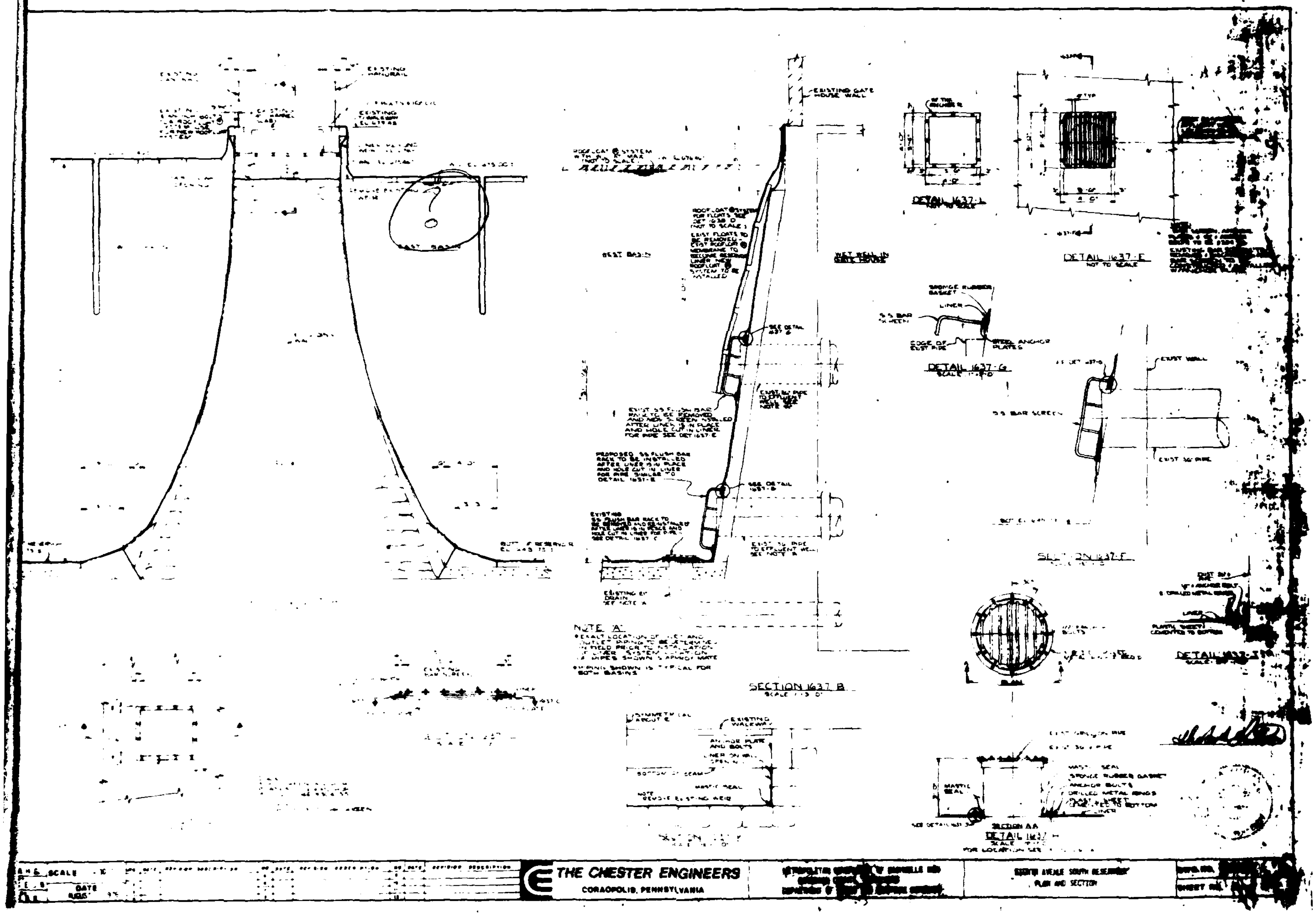
DEPARTMENT OF WATER AND SEWERAGE SERVICES

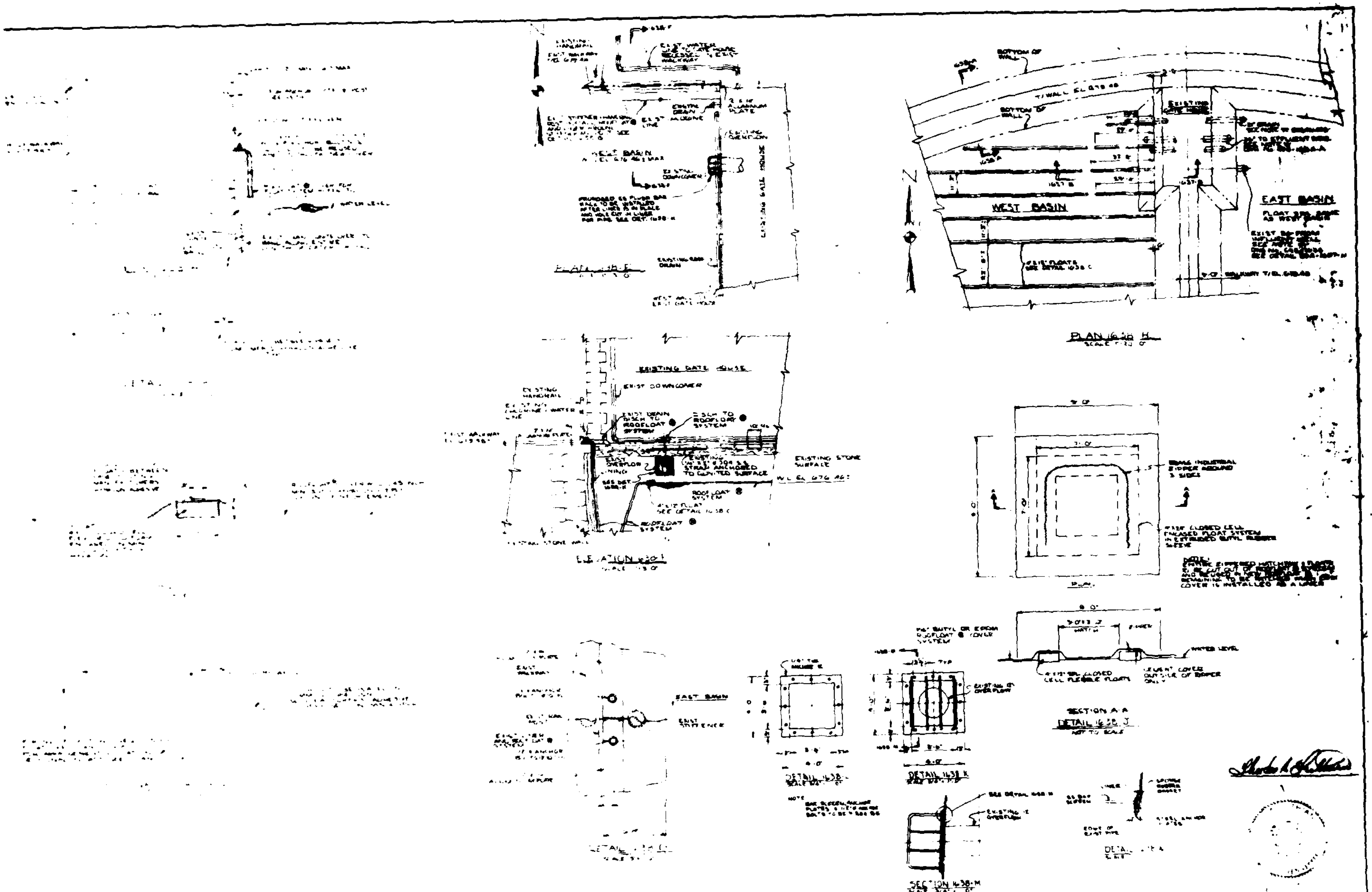
DWG NO.

SHEET NO. of 12



SCALE	DATE	
THE CHESTER ENGINEERS CORAOPOLIS, PENNSYLVANIA	[illegible]	COVER PLAN
DWG. NO. [illegible]	SHEET NO. 3 of [illegible]	


EXISTING HANDRAIL
EXISTING GATE HOUSE WALL
EAST BASIN
WEST BASIN
WET WELL IN GATE HOUSE
SEE DETAIL 1637-G
SEE DETAIL 1637-B
EXISTING DRAIN SEE NOTE A
NOTE 'A'
PIPING SHOWN IS TYPICAL FOR BOTH BASINS
SECTION 1637 B
DETAIL 1637-L
NOT TO SCALE
DETAIL 1637-E
NOT TO SCALE
SPONGE RUBBER GASKET
LINER
S.S. BAR SCREEN
EDGE OF EXIST PIPE
STEEL ANCHOR PLATES
DETAIL 1637-G
EXIST WALL
S.S. BAR SCREEN
EXIST 30" PIPE
SECTION 1637-F
PLAN
EXISTING WALKWAY
ANCHOR PLATE AND BOLTS
MASTIC SEAL
MASTIC SEAL
SPONGE RUBBER GASKET
ANCHOR BOLTS
DRILLED METAL RINGS
SECTION AA
DETAIL 1637-H
THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA
PLAN AND SECTION

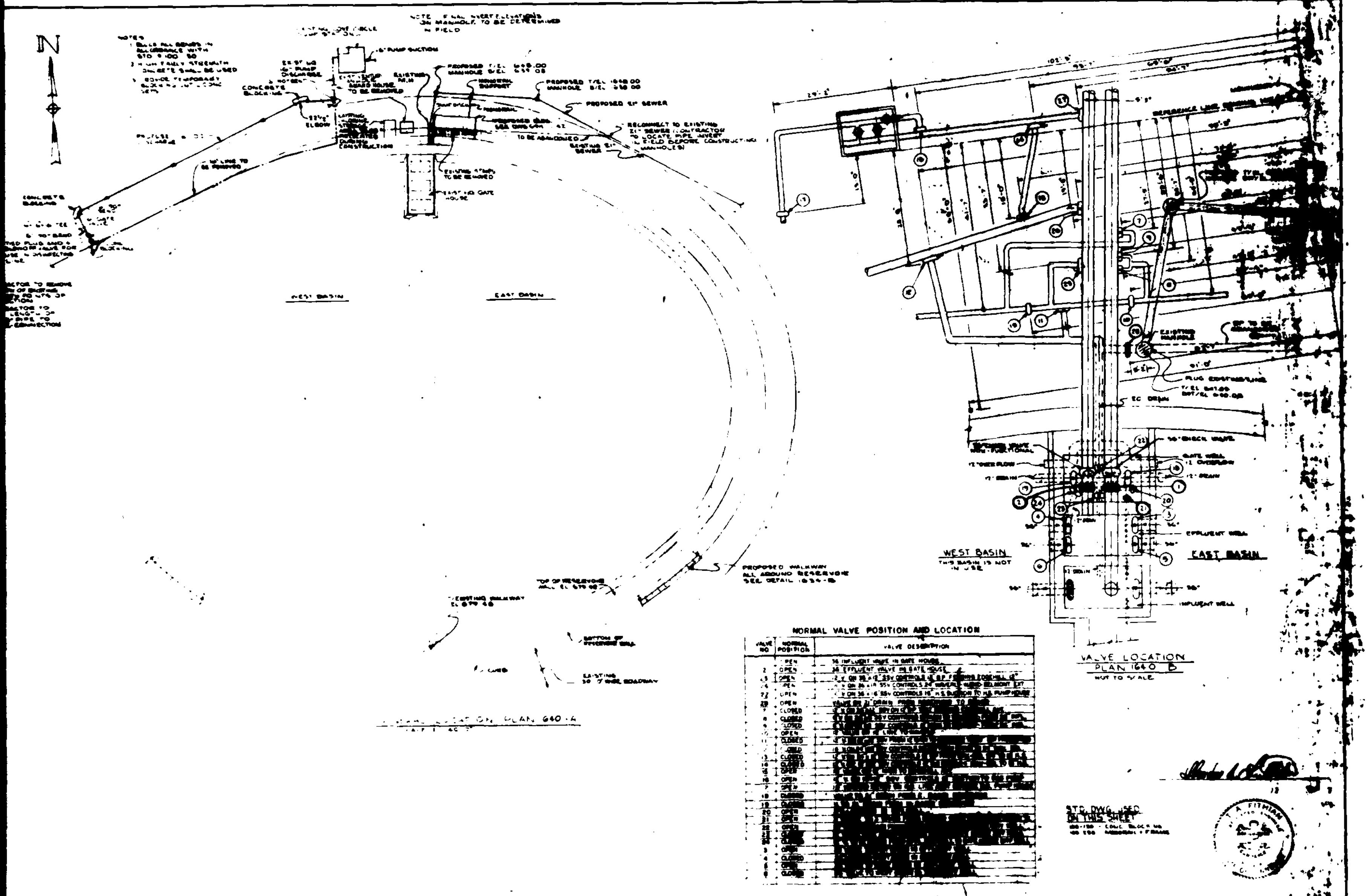


THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

METROPOLITAN GOVERNMENT OF NASHVILLE
DAVIDSON COUNTY, TENNESSEE
DEPARTMENT OF WATER AND SEWERAGE SERVICES

EIGHTH AVENUE SOUTH RESERVOIR
PLANS, SECTIONS & DETAILS

DWG. NO.
SHEET NO. 8 of 12



NORMAL VALVE POSITION AND LOCATION

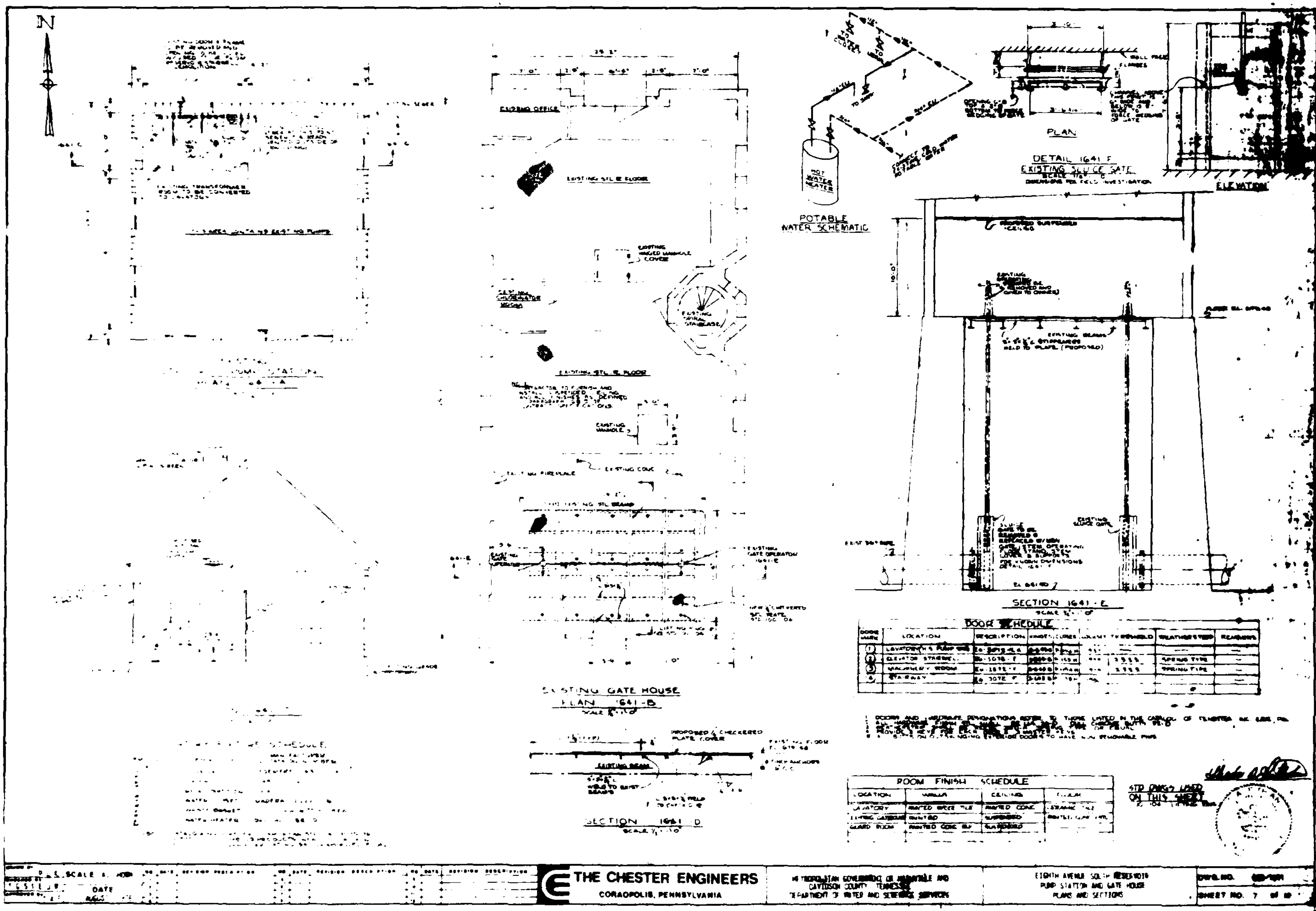
VALVE NO	NORMAL POSITION	VALVE DESCRIPTION
1	OPEN	36 INFLUENT VALVE IN GATE HOUSE
2	OPEN	36 EFFLUENT VALVE IN GATE HOUSE
3	OPEN	12 V ON 36 x 12 SSV CONTROLS 12 B.P. F[illegible] EDGEHILL 12"
4	OPEN	[illegible] ON 36 x 24 SSV CONTROLS 24 WAVERLY [illegible] EXIT
22	OPEN	[illegible] ON 36 x 16 SSV CONTROLS 16" H.S. SUCTION TO H.S. PUMPHOUSE
28	OPEN	VALVE ON 21" DRAIN [illegible] TO [illegible]
7	CLOSED	[illegible]
8	CLOSED	[illegible]
9	CLOSED	[illegible]
10	OPEN	[illegible]
11	CLOSED	[illegible]
12	CLOSED	[illegible]
13	CLOSED	[illegible]
14	CLOSED	[illegible]
15	OPEN	[illegible]
16	OPEN	[illegible]
17	OPEN	[illegible]
18	CLOSED	[illegible]
19	CLOSED	[illegible]
20	OPEN	[illegible]
21	OPEN	[illegible]
22	OPEN	[illegible]
23	[illegible]	[illegible]
24	[illegible]	[illegible]
3	OPEN	[illegible]
4	CLOSED	[illegible]
5	OPEN	[illegible]
6	CLOSED	[illegible]

SCALE AS SHOWN
DATE AUGUST 1976

THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

GENERAL LOCATION AND VALVE LOCATION PLANS

DWG. NO.
SHEET NO.



DOOR SCHEDULE

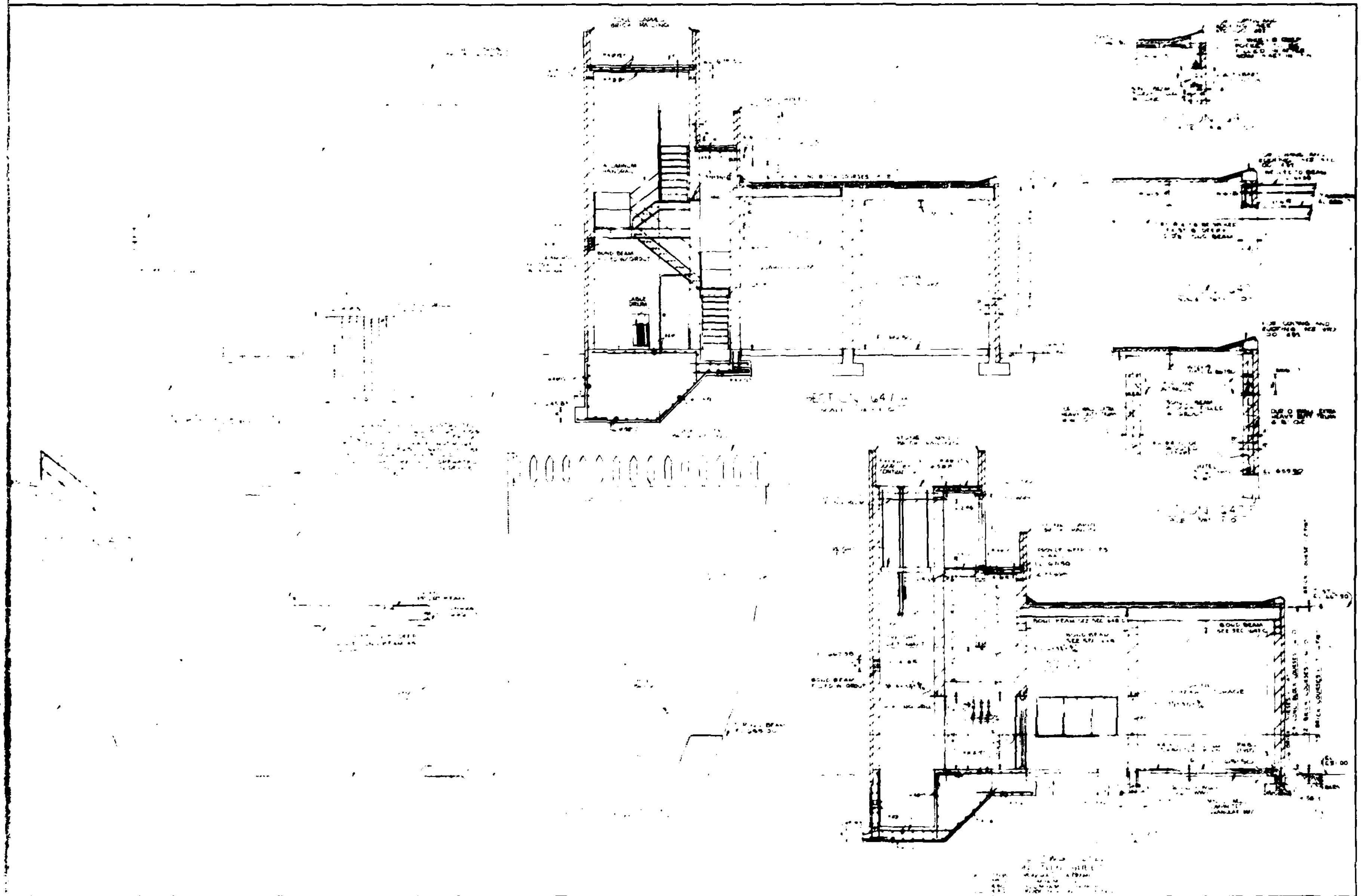
DOOR MARK	LOCATION	DESCRIPTION	HINGES	CLOSER	LOCKSET	THRESHOLD	WEATHERSTRIP	REMARKS
1	LAVATORY (?) PUMP RM.	[illegible]	[illegible]	[illegible]	[illegible]	—	—	—
2	ELEVATOR STAIRWELL	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	SPRING TYPE	—
3	MACHINERY ROOM	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	SPRING TYPE	—
4	STAIRWAY	[illegible]	[illegible]	[illegible]	[illegible]		—	—

1. DOORS AND HARDWARE DESIGNATIONS REFER TO THOSE LISTED IN THE CATALOG OF [illegible]
2. ALL HARDWARE FINISH [illegible]
3. [illegible]
4. PROVIDE 3 KEYS FOR EACH [illegible]
5. [illegible] BUTTS ON OUTSWINGING EXTERIOR DOORS TO HAVE NON REMOVABLE PINS

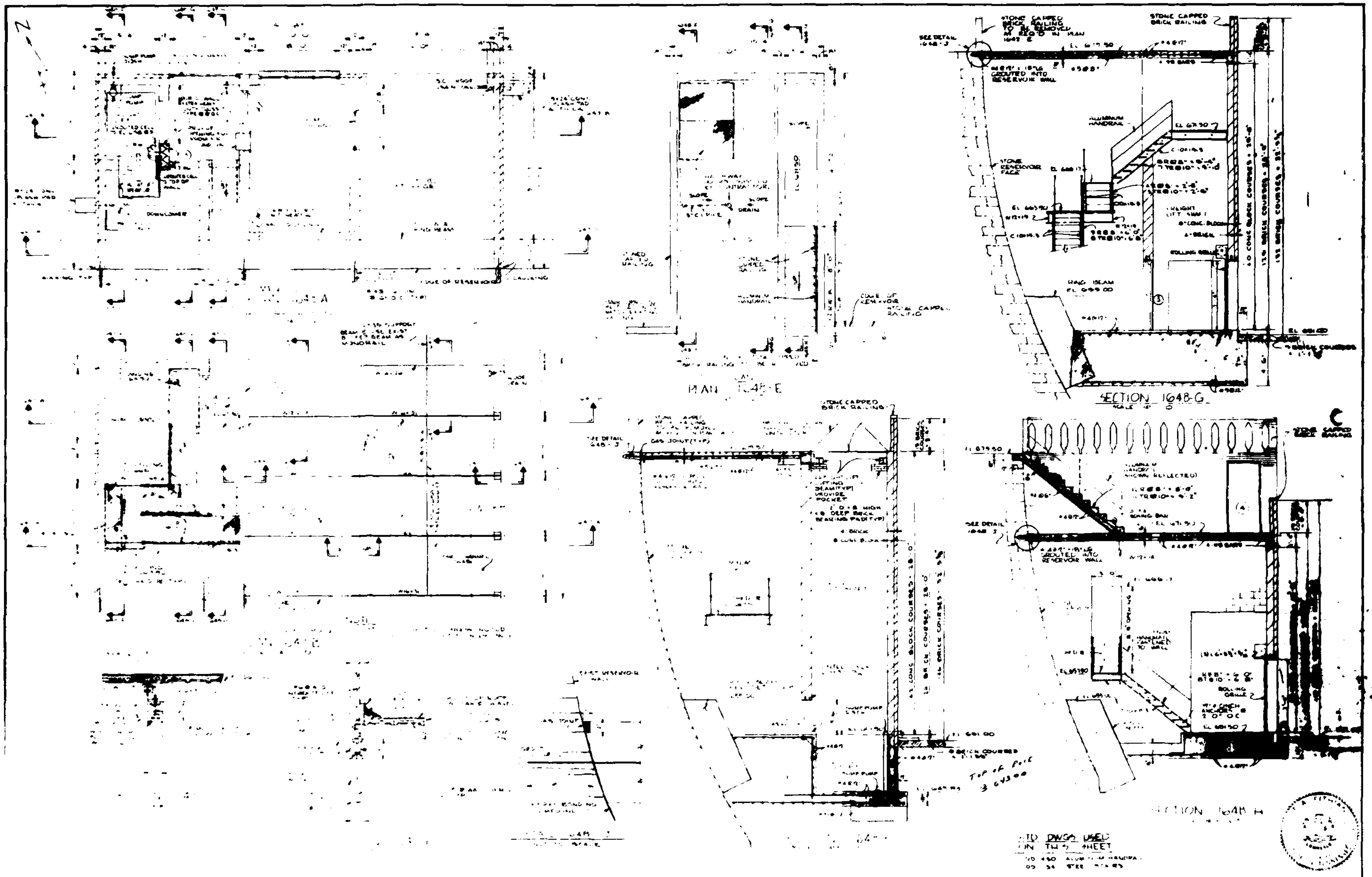
ROOM FINISH SCHEDULE

LOCATION	WALLS	CEILING	FLOOR
LAVATORY	PAINTED GLAZED TILE	PAINTED CONC.	CERAMIC TILE
EXISTING [illegible]	PAINTED	SUSPENDED	PAINTED CONC. [illegible]
GUARD ROOM	PAINTED CONC. BLK.	SUSPENDED	—

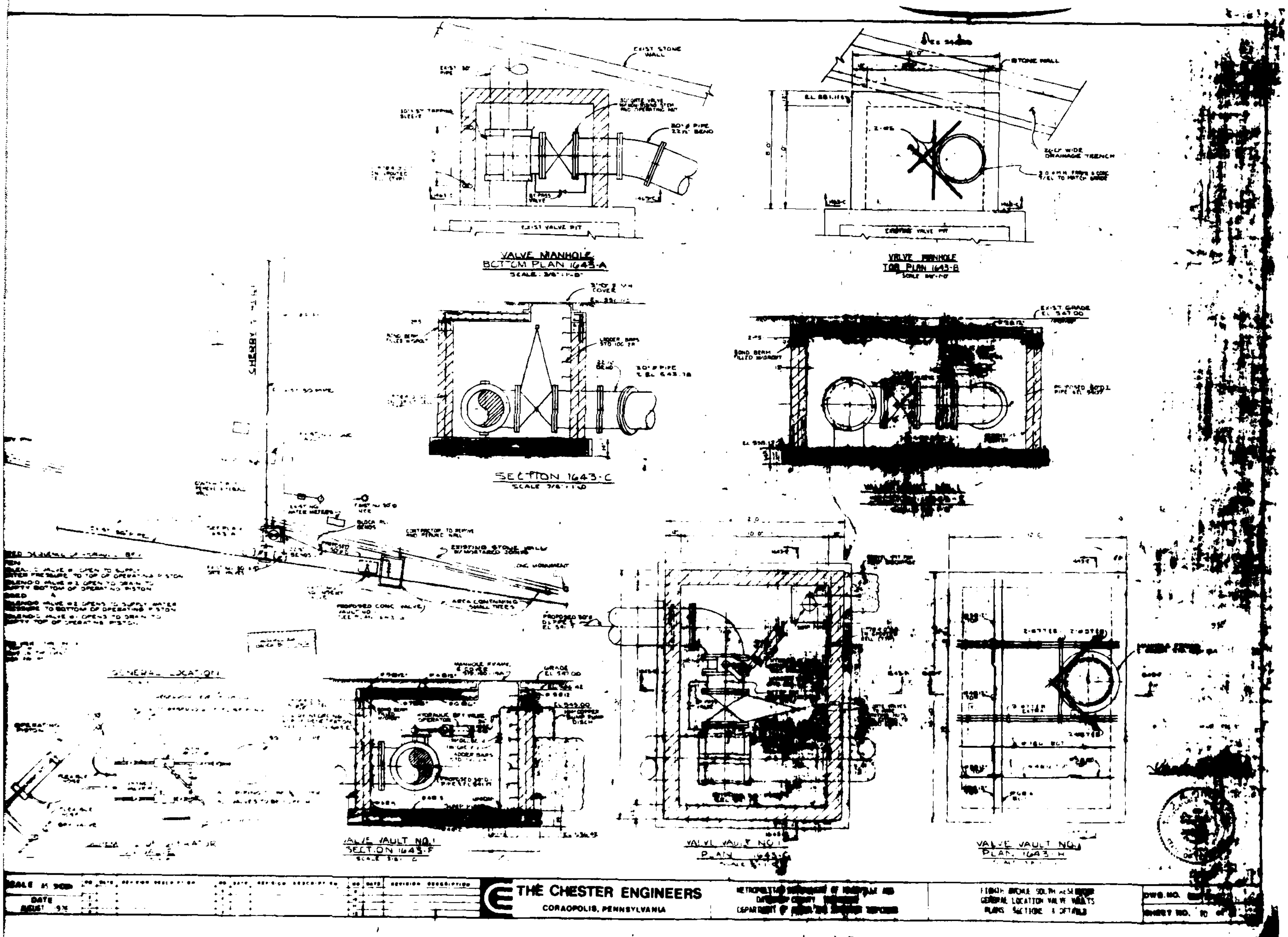
METROPOLITAN GOVERNMENT OF NASHVILLE AND DAVIDSON COUNTY TENNESSEE
DEPARTMENT OF WATER AND SEWERAGE SERVICES

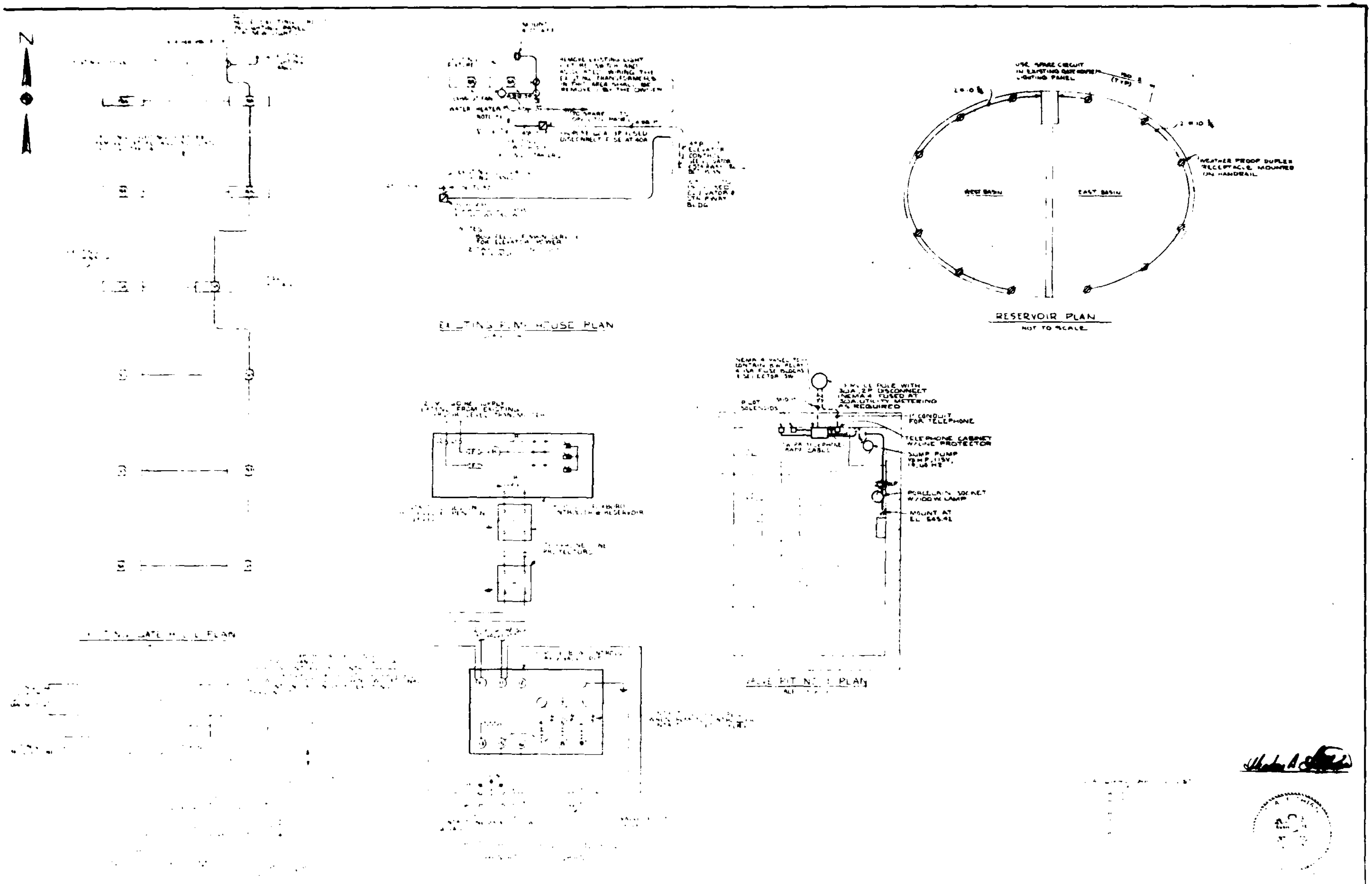


SCALE	THE CHESTER ENGINEERS		DWG NO 698-164
DATE	CORAOPOLIS PENNSYLVANIA		SHEET NO 8 OF 2



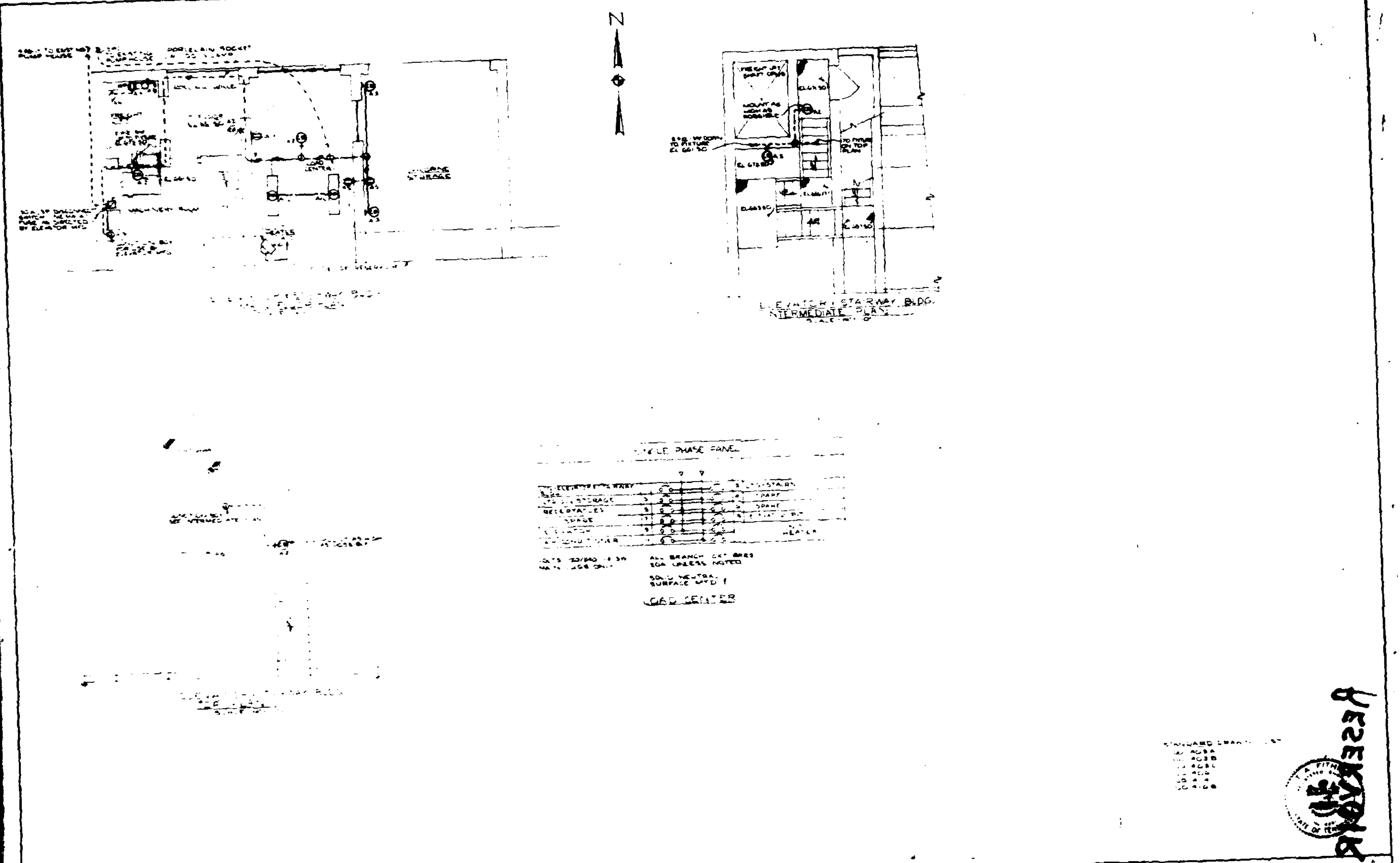
THE CHESTER ENGINEERS CORAOPOLIS PENNSYLVANIA	METROPOLITAN GOVERNMENT OF NASHVILLE AND DAVIDSON COUNTY TENNESSEE DEPARTMENT OF WATER AND SEWERAGE SERVICES	EIGHTH AVE. SOUTH RESERVOIR ELEVATOR AND STAIRWAY PLAN AND SECTION	DWG. NO 598-1998 SHEET NO. 6 OF 12


VALVE MANHOLE
BOTTOM PLAN 1643-A
SCALE: 3/8"=1'-0"
VALVE MANHOLE
TOP PLAN 1643-B
SECTION 1643-C
SCALE 3/8"=1'-0"
VALVE VAULT NO. 1
SECTION 1643-F
VALVE VAULT NO. 1
PLAN 1643-G
VALVE VAULT NO. 1
PLAN 1643-H
GENERAL LOCATION
EXIST. STONE WALL
EXIST. VALVE PIT
STONE WALL
EXISTING STONE WALL
CHERRY
THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA
DWG. NO.
SHEET NO.



THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

SCALE
DATE
DWG NO
SHEET NO



THE CHESTER ENGINEERS
CORAOPOLIS, PENNSYLVANIA

METROPOLITAN GOVERNMENT OF NASHVILLE AND DAVIDSON COUNTY, TENNESSEE
DEPARTMENT OF WATER AND SEWERAGE SERVICES

EIGHTH AVENUE SOUTH RESERVOIR
ELECTRICAL PLANS AND DETAILS

SCALE AS SHOWN

SHEET NO. 12 OF 12

APPENDIX F

HYDRAULIC AND HYDROLOGIC ANALYSIS

HYDROLOGIC AND HYDRAULIC ANALYSIS

According to OCE guidelines, 8th Ave. Reservoir must be able to safely pass the ½ to full Probable Maximum Flood (PMF). Six-hour rainfall depths for the Probable Maximum Precipitation (PMP) and the 100-year rainfall were obtained from the U.S. Weather Service's Technical Paper 40.

The only inflow to the reservoir during the six-hour storm is the rainfall itself or 29.3 inches for the PMP. The minimum distance between the top of the walkway and the reservoir cover is 3 feet. Therefore, the reservoir would not be overtopped by the full PMP.

APPENDIX G

CORRESPONDENCE

DEPARTMENT OF THE ARMY
NASHVILLE DISTRICT, CORPS OF ENGINEERS
P. O. BOX 1070
NASHVILLE, TENNESSEE 37202

IN REPLY REFER TO

ORNED-G

Mr. K. R. Harrington, Director
Department of Water and Sewerage Services
18th Floor - Stahlman Building
211 Union Street
Nashville, TN 37201

Dear Mr. Harrington:

As provided under authority of the National Dam Inspection Act, Public Law 92-367, all non-Federal dams in Tennessee must be inspected for the purpose of protecting life and property. The term "dam" includes any artifical barrier which impounds or diverts water. The 8th Avenue Reservoir qualififes as a "dam" under this definition and must be inspected.

A tentative date for the inspection has been set for 19 May 1981. However, in order to perform a thorough investigation, it is necesssary to review past design calculations, construction drawings, instrumentation data, etc. A portion of this material has been furnished to this office; however, additional material is needed. Specifically, material concerning design calculations for the tendons and information on the instrumentation that was installed would be helpful in assessing the stability of the structure. Any additional information would also be helpful. Please forward this material to Mr. Paul Bluhm of our Geotechnical Branch.

If you have any further questions regarding what material is needed or on any aspect of the inspection please contact Mr. Bluhm at 251-7366.

Sincerely,

E. C. MOORE
Chief, Engineering Division

NON-FEDERAL DAM INSPECTION REVIEW BOARD
PO BOX 1070
NASHVILLE, TENNESSEE 37202

ORNED-G

Commander, Nashville District
US Army, Corps of Engineers
PO Box 1070
Nashville, TN 37202

1. The Interagency Review Board, appointed by the Commander on 19 June 1981, presents the following recommendations after meeting on 3 September 1981, to consider the Phase I investigation report on 8th Avenue Reservoir located in Nashville, Tennessee.

2. The Board is in agreement with other report conclusions and recommendations following minor revisions.

FRANK B. COUCH, JR.
Chief, Geotechnical Branch
Chairman

WILEY B. SCOTT
Assistant Design Engineer
Alternate, Soil Conservation Service

EDMOND B. O'NEILL
Alternate, Division of Water Resources
State of Tennessee

THOMAS ALLEN
Hydraulic Engineer
Alternate, Hydrology and Hydraulics Branch

EDWARD B. BOYD
Hydrologic Technician
Alternate, US Geological Survey

L. E. LOCKETT
Structural Engineer
Alternate, Design Branch

DEPARTMENT OF THE ARMY
NASHVILLE DISTRICT, CORPS OF ENGINEERS
P. O. BOX 1070
NASHVILLE, TENNESSEE 37202

IN REPLY REFER TO

ORNED-G

28 AUG 1981

SUBJECT: Report of Phase I Investigation of 8th Avenue Reservoir, Nashville, Tennessee

Commander, Ohio River Division
ATTN: ORDED-T (Griff Ray)

1. Inclosed are three copies of our draft report covering the Phase I investigation of 8th Avenue Reservoir in Davidson County, Tennessee

2. The report is still in draft form at this time. Request return of copy containing color photographs along with your comments. We will furnish you a final version of the report when it is completed.

FOR THE COMMANDER:

1 Incl
As

E. C. MOORE
Chief, Engineering Division
For

ORDED-T (28 Aug 81) 1st Ind
SUBJECT: Report of Phase I Investigation of 8th Avenue Reservoir, Nashville, Tennessee

DA, Ohio River Division, Corps of Engineers, P.O. Box 1159, Cincinnati, OH 45201 17 September 1981

TO: Commander, Nashville District, ATTN: ORNED-G

1. The subject report needs revision to cover the following items.

a. The project should not be called deficient because of spalled and deteriorated concrete sidewalks. Deterioration of the sidewalks does not affect the integrity of the retaining structure.

b. If the structure is to be classified as deficient because of the seepage in the gate house, the report should state what danger is created by that seepage.

c. Para 3-5-1h. This paragraph indicates that the structure is deficient because of the items discussed above, but at the same time, this paragraph and other paragraphs indicate that the structure does not have any apparent safety or structural deficiencies. These conflicting statements must be resolved.

d. The report indicates that the seepage through the reservoir retaining wall is not serious. Also, it is stated that the deteriorated stone condition is not serious. At the same time, the trip reports in Appendix D have conflicting statements regarding the construction of the walls. One trip report indicates that the walls contain rubble fill; another report indicates cyclopean concrete; and another report speaks of impervious clay. Until one knows the details of the walls, it is impossible to assess the impact from the seepage and the deteriorated stone condition. The report should not contain an indication that the seepage condition and the deterioration of the stone is not serious. Consideration should be given to recommending that the owner obtain an A/E firm to investigate the condition of the walls.

e. The need for an A/E to investigate the structural condition of the masonary wall and the effects of the tie-back ring beam was previously suggested in the report by Shannon and Wilson. This recommendation should be brought to the owners attention.

f. A recommendation should be made to encourage the owner to continue the visual inspections presently being made.

2. General.

a. Para 2-4-1, Main Report. The third sentence of this paragraph describing the local geology should be reviewed for accuracy. It appears that this discussion is related to bedding instead of jointing.

ORDED-T (28 Aug 81) 1st Ind 17 September 1981
SUBJECT: Report of Phase I Investigation of 8th Avenue Reservoir, Nashville, Tennessee

b. The table of contents indicates text page numbers, but the pages are not numbered.

c. The text contains statements which conflict with information furnished in the Appendices.

d. Appendix B. North arrows should be added to the maps and sketches.

3. Appendix B, Section B.B and Appendix D, Project History. The text, para 2-2, indicates that the 8-foot wall was not installed around the entire reservoir. This conflicts with Appendices B and D and should be corrected.

FOR THE COMMANDER:

Richard C. Armstrong
RICHARD C. ARMSTRONG
Chief, Engineering Division

APPENDIX H
PREVIOUS INVESTIGATIONS

BEVERLY BRILEY, MAYOR

Metropolitan Government of Nashville and Davidson County

K. R. HARRINGTON
DIRECTOR OF WATER AND SEWERAGE SERVICES

DEPARTMENT OF WATER AND SEWERAGE SERVICES
WATER TREATMENT PLANT
OMOHUNDRO DRIVE
NASHVILLE, TENNESSEE 37210

May 27, 1975

MEMO

TO: Mr. K. R. Harrington
Director - Water & Sewerage Services

FROM: Mr. Lester Williams, Jr.
Chief Operational Engineer

IN RE: 8th Avenue Reservoir Leak 75-W-46

The following is a chronological record of the events relating to the leak discovered in the 8th Avenue Reservoir on Wednesday, May 21, 1975.

At approximately 12:30 p.m. Mr. V. W. Frye contacted me regarding a leak at the 8th Avenue Reservoir. After learning of the description of the leak, Mr. F. L. Clinard and myself proceeded to the reservoir to investigate. Upon arriving, we observed seepage in the northwest quadrant of the reservoir, beginning at a point due west and extending for about 50° to the north and covering approximately 100 yards. The seepage was confined to the lower four courses of stone. The worst leak was in the middle above the 4th course of stone with other noticeable seepage the entire span described, tailing to the 1st course of stone at the edge in both directions. The water quantity was sufficient enought to pass through a gravel bed and across the paved road.

At this point I put in a call for Mr. W. F. Brock and/or Mr. K. R. Harrington. While waiting for their return call, we proceeded to contact the engineering survey party to recheck level measurements at the reservoir, also, flow measurements were begun at the worst leak described above, because it appeared that the leak was increasing.

The 1st measurement of the leak was 1 quart per 50 seconds. This quantity was checked periodically at 15 minute intervals with no changes during the afternoon.

Mr. K. R. Harrington returned my call at approximately 1:30 p.m. and at which time I informed him of our findings and he suggested the possibility of beginning the draining of the west basin and agreed instead to cut off the influent to the west side and lower the level by using the water. This was done while the above precaution measurements continued.

Mr. K. R. Harrington and Mr. W. F. Brock arrived at the reservoir shortly after 2:00 p.m. and no noticeable change had been observed at this time. It was decided to lower the west basin during the afternoon and night as low as usage would allow and then valve off. The seepage was to be continuously monitored during the night for any changes. A meeting was set for the following morning at 9:30 a.m. with Mr. T. A. Fithian of Chester Engineers and Mr. Ray Throckmorton of Geologic Associates. Another level survey was to be run early in the morning.

During the evening hour the reservoir level was lowered to approximately 8' below the overflow. By 6:00 p.m. the seepage had noticeable decreased and by 10:00 p.m. there was almost no flow through the monitored seepage point.

Neither of the level surveys showed any significant deflections.

By 9:00 a.m. on Thursday, May 22, 1975, the seepage had decreased to no more than that observed in prior years.

At a meeting on the premises at 9:30 a.m., the following persons were present, Mr. V. W. Frye, Mr. L. L. Williams, Mr. W. F. Brock of the Water and Sewerage Department, Mr. T. A. Fithian, Mr. Harold Kurtz of Chester Engineers, Mr. Ray Throckmorton of Geologic Associates and Mr. Jerry Free of Pressure Concrete. At that time all the information was reviewed and it was elected to continue all observations. In addition, Mr. Throckmorton and Mr. Fithian were instructed to make an analysis of the situation, also, Mr. Throckmorton was to set up a plan for taking some sample core drills. All parties were to be avised of any changes monitored.

By Thursday afternoon almost all seepage had stop above the 1st course of stone.

The reservoir level was maintained at 8' below overflow in the west basin.

On Friday, May 23, 1975, the seepage continued to dry up with only a few noticeable wet spots. Continuation of monitoring remained in affect as before.

Subsequent happenings and events relating to the leakage are recorded in a log which is maintained by Mr. V. W. Frye.

Respectfully submitted,

Lester Williams, Jr.

Lester Williams, Jr.
Chief Operational Engineer

LLW:jdb

cc: Mr. W. F. Brock, Jr.
Assistant Director - Water & Sewerage Services

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

Reply to
FRANKLIN, TN 37064
615—794 3596

May 30, 1975

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
8th Floor, Stahlman Building
211 Union Street
Nashville, Tennessee 37201

Attention: Mr. K. R. Harrington

Gentlemen:

Re: Stability and Leakage Study
Main Reservoir, 8th Avenue, South
Nashville, Tennessee

Regarding the above site and our inspections and various conversations regarding it, we expect to begin the subsurface exploration at once and will keep you informed of its progress. Initially, three or four holes, rather widely spaced across and beyond the visibly affected area, will be cored to depths which will permit positive correlation of the strata and identification of any potential planes of slippage. Most of the holes will be vertical; some inclined drilling may be necessary. These exploratory holes, and some intermediate holes as well, will be drilled and permanently cased into bedrock in order to observe the static water level.

In order to properly assess the subsurface data and its relationship to the foundation of the reservoir, it will be necessary to excavate some test pits adjacent to the foundation. These pits will be dug,

6/2/75 - Copy to Bill Brooks, Buddy Williams, Ted Fitton
Coach Johnson and Mike Patton

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Page Two
May 30, 1979

one at a time, normal to the arc of the wall and will extend to such depths as may be necessary to log and identify the footing condition, contact elevation, etc. When completed the pits will be backfilled to grade with plant-mix 3,000# concrete.

In order to properly evaluate the data to be gathered, we need copies (or loan of the originals) of all pertinent drawings, reports and other documents pertaining to the design, construction and repair of the facility over the years. Mr. Williams has indicated that a topographic map of the reservoir areas - extending well downhill into adjacent properties - will be prepared soon. These data are essential to the stability study, too, as we will drill lines of probings to define the weathered bedrock surface outward from each cored hole location.

We expect to proceed with caution and will keep you informed of our progress and findings. In the writer's absence (until June 15th) our work will be under the direction of Mr. John Hageman.

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Page Three
May 30, 1975

At this time there is no way for us to estimate the ultimate cost of this study. Our charges will, however, be based on the appended fee schedule. We appreciate this opportunity to be of continuing service to you.

Yours very truly,

GEOLOGIC ASSOCIATES, INC.

Ray Throckmorton

R. T. Throckmorton, Jr., P. E.

RTT/mr

Enclosure - Fee Schedule

Copy - Mr. Ted Fithian, Chester Engineers

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

Reply to
FRANKLIN, TN. 37064
615--794-3596

July 30, 1975

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee 37201

Attention: Mr. William Brock

Gentlemen:

Re: Interim Report,
Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

The Eighth Avenue Reservoir structure (Kirkpatrick Hill) has been repaired at least twice previously. We have not been supplied with any data relating to the catastrophic failure of 1912, except that gleaned by inference from the 1921 data, but we understand that the affected area encompassed an arc at the southeast segment of the east basin. Presumably, the wall was displaced outward along planes of weakness in the subfoundation. We note that the test pits excavated along the 1912 failure area, during the 1920 study, showed that the footings in the former area, as reconstructed, extended to depths of ten or more feet below ground level. No doubt the reconstruction extended through the weathered rock zone to sound strata. We point

BRANCHES. P. O. BOX 9278 KNOXVILLE, TN 37920 615—573-7383
2711 FT. CAMPBELL BLVD. HOPKINSVILLE, KY. 42240 502—886 0721

Metropolitan Government of
  Nashville and Davidson County
Page Two
July 30, 1975

out that the 1912 area is located at approximately 180° to the presently affected reach of the wall, which may imply that some of the subsurface conditions found during the present study are related to the causes of the 1912 episode.

The foundation remedial work performed in 1921 consisted essentially of underpinning a further segment of the east basin after a horizontal crack was found in the interior face. The history of this work is well-documented and has been valuable during the present study.

We are told that the subject area (north-northwest arc of the west basin) has displayed evidence of seepage for some years. While the visible leakage was not alarming nor objectionable, it was carefully monitored, especially in the light of the two previous failures which have occurred in the structure. In the latter part of 1974, cleaning of the basin - for the first time in about forty years - was completed, along with some other construction efforts, such as guniting the walls, etc. The basin was put back in operation during March 1975. In the late spring seepage increased in volume over any occurring previously and extended as high as four or five masonry courses above ground level. Initial observations seemed to indicate some diurnal influence in that flows increased during the afternoon hours when the sun's rays bore most directly on that arc. Flows decreased or ceased altogether during the evening and morning hours.

On Thursday, May 22, 1975, the writer first inspected the site in the company of Messrs Brock, Williams, Fithian, Kurtz and others. We were authorized to begin an exploratory program directed toward determining subsurface conditions adjacent to the visibly affected area.

EXPLORATION AND TESTING.

Eleven holes were drilled at locations shown on the Plan, Sheet 1. Ten of these were drilled vertically to depths which permitted assessment of bedrock degradation and positive geologic correlation. One hole was drilled at an inclination of 45° in order to further study the bedrock structure. Permanent casing was placed in most of the holes and the static water level in each has been monitored by us, as well as Metro personnel. The bedrock cores are stored at our Franklin office. All of the bedrock cores were logged in detail, representative samples of the clay seams in the bedrock cores were tested for natural moisture content, particle size distribution and their Atterberg limits determined. The basic data and our interpretation of it are shown on the appended Profiles.

Metropolitan Government of
Nashville and Davidson County
Page Four
July 30, 1975

SUBSURFACE CONDITIONS.

The existing ground surface surrounding the reservoir is undoubtedly a much-modified version of the original landscape. We suspect that much of the soil and weathered rock slabs excavated during the original construction - and perhaps by later repairs - were spread on the slopes surrounding to construct the present benches and slopes. Therefore, the existing "overburden" consists of an uncontrolled mass of soil and rock slabs overlying the original topsoil and soil-weathered rock profile.

Bedrock at the site consists of thin to thick bedded shaly limestone of the upper portion of the Catheys Formation. Degradation of the bedrock has extended to depths averaging nearly 12 feet below the existing ground surface. Locally, severe degradation, apparently related to deformation of the strata extends to twenty or more feet. Weathering of the strata has included lateral opening, staining and leaching of bedding places following the dip of the strata, as well as similar degradation along high angle tension fractures which crisscross the site. We suspect that the prominent jointing is aligned along

northeast-southwest and northwest-southeast azimuths. Joint frequency may be on the order of ten feet, or less. Further, we have found additional evidence of bedrock faulting which is probably not the same fault described in the 1920-21 work. This revelation is not surprising as many minor faults have been discovered in the past and by recent exploration and construction in the strata underlying the city. Several faults which seem to trend across St. Cloud's Hill are exposed to the northeast of the reservoir in the right of way cut for I-65. The attitude, displacement and degree of weathering displayed by these latter faults seems to agree well with those found at the reservoir. We recommend them as good examples of conditions probably existing in the subgrade for the reservoir. While such faults have been active in the geologic past, they are now construed as inactive in that movement is not likely to occur along them, even in the event of seismic events in the area. Parenthetically, Nashville is located within Seismic Risk Zone I; however, it is "sandwiched" between Zone II contours to the east and west. Assessing the stability of such a structure under seismic loading is, at best, an exercise in empirical arithmetic.

Metropolitan Government of
Nashville and Davidson County
Page Six
July 30, 1975

Static water levels recorded in the drill holes indicate a considerable slope in the phreatic surface, both normal to the reservoir and parallel to the arc. Some of the discrepancies may be attributed to the relative water-tightness of the bedrock at the hole, while others, no doubt, reflect the actual phreatic level. It is significant that the valid water levels responded almost immediately when the effect of the west basin static head was essentially removed from the groundwater system.

CONCLUSIONS AND RECOMMENDATIONS.

The structure was built nearly a century ago according to founding standards and criteria far less stringent than those in use today. Two previous failures, the last some 55 years ago, were, as best we can tell, entirely due to inadequacies in the subgrade of the structure. In each instance, only the segment affected was repaired. The records indicate that both times the foundation was reconstructed on essentially unweathered bedrock at levels considerably lower than the original.

Our analysis of the data we have gathered indicates that this segment of the reservoir is undoubtedly existing in a precarious state of stability. All of the structural and subsurface parameters contribute to this condition; none negate it.

The masonry footings rest on severely degraded bedrock which contains residual clay seams of medium to high plasticity; i.e., fat, slippery clays - whose shear strength is substantially lessened when their moisture content is increased.

The west reservoir basin is leaking - somewhere - and contributing a constant head and supply of water to the soil-rock system.

The bedrock strata have a component of dip downward and outward from the affected arc. In addition, one or more faults transect the bedrock adjacent to, and probably extend under, the reservoir wall. These faults are significant primarily because bedrock weathering has been intensified along and adjacent to them.

A stability analysis based on the data at hand indicates that the foundation for the masonry wall is in a marginal state of stability and further changes in the physical characteristics of the soil-rock system; i.e., an increase in the moisture content of the clays, further piping of materials by the flowing groundwater, seismic events, etc., could cause a catastrophic failure if the full reservoir head were imposed at the time. Separate comments regarding the stability analysis are appended.

The options at this point are numerous and range from continuing to operate the reservoir in a normal manner to abandoning the structure. In order to recommend further actions, it is essential to know what the projected life of this facility is based on its necessity to the operation of the system, the amortization of recent sums spent on it, and so on.

Assuming that the reservoir must remain a part of the Metro water system for the foreseeable future, we are ready to discuss what steps may be taken to restore its integrity and what information may be necessary to the design of the remedial construction.

Respectfully submitted,

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton, Jr., P.E.

RTT/mr

Enclosures - Appendix A
Laboratory Data
Drawings

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

Reply to
FRANKLIN, TN 37064
615—794-3596

EIGHTH AVENUE RESERVOIR STABILITY ANALYSIS

PROJECT NO. 75-090

APPENDIX A

A general stability analysis of the foundation for the Reservoir was performed using the Wedge Analysis Method. Forces imposed by the structure and the surcharge of the fluid head were resolved based on the available structural data and the geometry of the foundation. The configuration of the 1920 failure as recorded by The Chester Engineers was considered in selecting the boundary for the active earth pressures. Potential failure surfaces were selected after studying the weaknesses in the subfoundation materials recovered by the core drilling program. Physical properties of the soil-rock system were estimated using basic laboratory test data.

Owing to the nature of the subfoundation materials, significant variations in the soil-rock properties occur within the zone adversely affected by the structural load. The variations which are most likely to affect the stability are lateral or planar features - namely nearly horizontal, continuous layers of clay. The analysis shows that minor variations in the strength of the clay, as well as the position of the phreatic surface, have a substantial effect on the stability of the foundation and surrounding earth mass. For example, in some cases the

BRANCHES: P. O. BOX 9278 KNOXVILLE, TN. 37920 615—573-7383
2711 FT. CAMPBELL BLVD. HOPKINSVILLE, KY. 42240 502—886-0721

margin of safety for stability is increased by 30% if the phreatic surface is lowered below the potential failure surface. Similarly, variations in the soil's shear strength on the order of 700 PSF can cause a 50% variation in the safety factor. Furthermore, the analysis indicates that a 50% reduction in the Reservoir head would result in an increase of 40% in the stability safety factor. While the above analysis is useful in assessing the operational alternatives and the risks, it must be recognized that the data is based on a conservative selection of potential failure surfaces. In actuality, we cannot be certain if, and along what planes, the instability is occurring. Determination of the position of the potential failure surface is essential to accurately assess the stability. Moreover, methods of remedial treatment will depend on the limits and nature of the subsurface movement. These facts can be determined only by monitoring the dynamic behavior of the subsurface by means of inclinometers installed at selected locations around the perimeter of the Reservoir.

Based on the stability analysis, we conclude that under the influence of the full Reservoir surcharge, the stability margin of safety for the subject segment of the wall is on the order of 1.1. Over the

long term, and sustained full reservoir conditions the phreatic surface within the underlying and adjacent soil system will continue to slowly raise the moisture content of the clays, thereby further reducing their shear strength.

GEOLOGIC ASSOCIATES, INC.

GEOLOGISTS AND ENGINEERS

Reply to:
FRANKLIN, TN.
615—794-35

August 1, 1975

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee 37201

Attention: Mr. K. R. Harrington, Director

Gentlemen:

Re: Foundation Investigation
and Stability Studies,
Phase II, Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

We have completed our initial investigation of the subject site and have discussed it in detail with your staff and consulting engineers. At the conclusion of the conference held July 30, 1975, we were directed to prepare a proposal outlining a program of exploration which will define subsurface conditions adjacent to the entire perimeter of the structure. Further, portions of the exploratory program will be adapted and equipped to permit monitoring minute movements in the sub-foundation materials. The above two-part program will proceed essentially concurrently.

We have set-up a tentative exploratory program consisting of cored holes located at about 100 feet chords immediately adjacent to the footing system; these will be located in a fashion similar to

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NATIONAL PROGRAM OF INSPECTION OF NON-FEDERAL DAMS, TENNESSEE. --ETC(U)
SEP 81 P F BLUHM DACW62-81-C-0056

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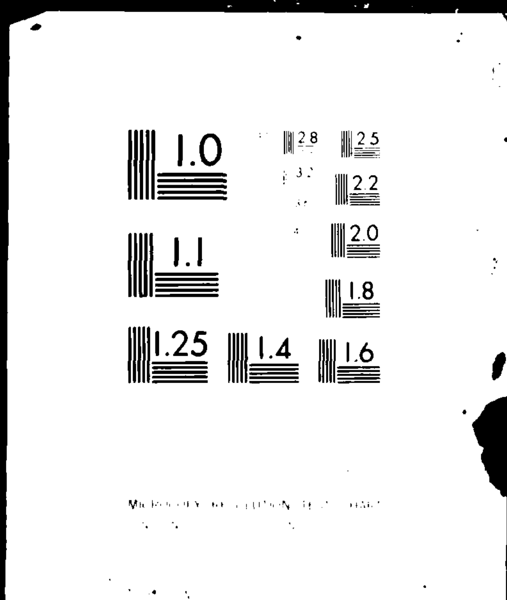


Holes 10 and 11 of the Phase I work. In addition, cored holes may be spaced on chords of 200 feet some 50 feet from the reservoir, as per Holes 1, 2, 3, etc. of the initial work. Finally, holes located at the quadrant points, as per Hole 4, may be needed. All of this work will be directed toward assessing the degree of bedrock weathering and the geologic structure, all of which affect the long-term stability of the reservoir. It will also enable us to confirm the efficiency of the repairs (underpinning) performed in years past. As an adjunct to the exploration, we expect to drill one hole from the top of the reservoir, through the wall, and into the sub-foundation. This hole will be located in the segment of wall arc through which the present seepage occurs. It will be utilized in studying the condition of the cyclopean concrete in the wall, for monitoring the level of any water encountered, etc.

The second part of the program will consist of adapting the drill holes for monitoring with an inclinometer. These locations will number about 15 and they will be spaced at about 200 feet centers on the reservoir perimeter, as well as at some distance away at the quadrant points for

Metropolitan Government of
Nashville and Davidson County
Page Three
August 1, 1975

use as "controls". We recommend the use of this system as the most sophisticated instrumentation available with which to monitor minute movements in the foundation materials. This procedure is the only reliable and efficient means for detecting a critical loss in stability prior to the occurrence of increments of movement sufficient to cause visible distress. Further, the monitoring program will provide data useful in designing any remedial construction.

We will also permanently install some tiltmeters on the reservoir rim as an added feature of the monitoring of the safety of the structure.

We have prepared a detailed in-house estimate of the cost of the program we have outlined; we will be happy to discuss it with you at your convenience. Naturally, the program will be subject to change as conditions encountered dictate. Most of the unit prices are included in our Standard Schedule of Fees, which you have on file. Adapting the 15 drill holes for permanent monitoring by inclinometer will be charged at the rate of $450 each. We have also included in this estimate about 150 hours (over a two-month period - ten readings each) of monitoring the indicators at $50/hour. The monitoring process

requires the use of two men plus the inclinometer and readout. The total of our estimate for performing all of the above described services is $41,850. We leave the assessment of the contingencies inherent in this program open to further discussion.

We appreciate this opportunity to continue the study, and having your verbal approval, expect to proceed with the exploration beginning Monday, August 4th. We will keep you and your consultants advised of conditions encountered.

Respectfully,

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton, Jr., P.E.

RTT/mr

Copy: Mr. T. A. Fithian
The Chester Engineers

GEOLOGIC ASSOCIATES, INC.

GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

October 9, 1975

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee 37201

Attention: Mr. K. R. Harrington, Director

Gentlemen:

Re: Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

The purpose of this letter is to provide you with a short statement regarding progress of the subject study.

Subsequent to the initial study which was confined to the northwest quadrant, we were directed to proceed with a detailed exploration of the entire structure. As of October 8th, we have suspended the subsurface exploration program and will shortly submit a comprehensive report detailing all of the basic data and our interpretation of it. Briefly, we expect that the report will confirm the supposition that the subfoundation materials are for the most part deeply weathered and possibly exist in a state of stability having a low factor of safety.

In order to properly provide for the safety of the structure, as well as the public, during the time interval required for further study, development of plans and execution of future remedial work, we have installed the most sophisticated monitoring system available. Fourteen inclinometers have been installed at the site. Ten of these are located around the perimeter of the reservoir; four are located elsewhere on the grounds. Further, "tilt plates" have been set on the top of the wall at the intersection of the axes. The monitoring of all these installations is in its initial phase and prolonged observation will be necessary. As soon as possible, and provided that the data is favorable, we may recommend that the west basin be returned to use - under close observation. These, and other decisions, will be made on a step-by-step basis with your concurrence and that of your other consulting engineers.

We are available to discuss the details of our work at your convenience.

Yours very truly,

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton, Jr., P.E.

RTT/mr

Copy: Mr. W. F. Brock
Mr. Lester Williams
Mr. John Upham

ENVIRONMENTAL
ENGINEERS & PLANNERS

5 Dunwoody Park, Suite 118
Atlanta Ga 30341
(404) 394-8620

THE CHESTER ENGINEERS

March 30, 1976

Mr. K. R. Harrington, Director
Department of Water & Sewerage Services
8th Floor - Stahlman Building
211 Union Street
Nashville, TN 37201

Dear Mr. Harrington:

Engineering Report
on the Stability of
The Eighth Avenue Reservoir

We are pleased to provide you with the accompanying report, prepared in collaboration with your consulting geologist, Geologic Associates, Inc.

The report concludes that the reservoir structure is in a marginal state of stability because of its subgrade of highly weathered rock, interspersed with clay seams.

The most feasible means of increasing the factor of safety against failure of the subgrade would consist of the installation of an instrument monitoring system, providing a flexible type water tight liner and construction of a tendon or tied-back ring system, anchoring the structure to sound bed rock.

Our current estimate of cost including the aforementioned construction together with miscellaneous structural and hydraulic improvements engineering, administration, inspection and construction contingencies is $1,960.000.

Prior to proceeding with the recommended remedial work, we would suggest that the structural integrity of the walls be investigated with several core holes. Such investigations should be helpful in the final development of the monitoring and tendon systems.

At such time as you and your staff have had the opportunity to review the report, we would be happy to meet with you to answer any questions and to discuss the report in detail.

Very truly yours,

THE CHESTER ENGINEERS

T. A. Fithian, P. E.
Regional Director

TAF/lh

cc: Geologic Associates

76 WD 47A
8th Ave Res.

ENGINEERING REPORT
ON STABILITY OF
THE EIGHTH AVENUE RESERVOIR

The Chester Engineers

March 1976

TABLE OF CONTENTS

ENGINEERING REPORT
ON
STABILITY OF THE EIGHTH AVENUE RESERVOIR

PURPOSE

The purpose of this report is to provide an engineering review of investigations and technical reports made on the stability of the 50 million gallon, Eighth Avenue Reservoir. The report will also present recommendations as to the most feasible method(s) to enhance the stability and to provide means of monitoring movement of the structure.

GENERAL

The Eighth Avenue Reservoir was constructed in 1889 and known at that time as the Kirkpatrick Hill Reservoir. The structure is elliptical in plan; the minor axis is 463.4 feet and the major axis is 603 feet, with a center wall across the minor axis. The tops of the walls are eight feet thick having an elevation of 676.5. The reservoir floor has an elevation of 642.75 and the walls at this level are approximately 23 feet thick.

A failure occurred in a portion of the easterly basin in 1912 which displaced about 200 feet of wall. Examination of photographs taken of the displaced wall leads to the conclusion that the foundation failed or slipped, allowing the bottom of the wall to move laterally outward. The wall was rebuilt in 1914 together with the construction of a perimeter french drain system, adjacent to and inside the east basin wall. At the

same time, the interior walls were gunited and the floor of the basins treated with asphalt and covered with concrete.

In June of 1920, inspection of the structure indicated additional problems in the east basin and it was removed from service. Further inspection of the interior indicated cracks in the masonry and a study complete with core holes and test pits was initiated. As a result of the study, approximately two-thirds of the affected wall was underpinned and another french drain constructed. As part of this work, the interior walls of both basins were scaled and a new waterproofing and gunite system applied. An additional concrete floor was installed over the 1914 floor.

To comply with State and Federal regulations, contracts were let in 1974 to install covers over the two open basins. The work included cleaning the reservoir bottoms, guniting the interior walls and the installation of a butyl rubber, floating cover. Work on the easterly basin was completed in 1974 and work on the westerly basin was completed in March of 1975 and the basin returned to operation. Some time in the Spring of 1975, seepage through the lower courses of masonry was noted along the northwest arc of the west basin. Since the seepage was greater than that which had been previously observed, it was decided to proceed with an exploratory program directed toward determining subsurface conditions adjacent to the seepage area. Geologic Associates were retained to make such exploratory investigations and the data obtained indicated that the subsurface materials are poor and do not provide an

adequately safe foundation for the structure. As a result, the west basin was drained.

Analysis of the data obtained from the exploratory investigations indicated that it would be prudent to investigate the foundation of the entire structure. Such work was performed and a report prepared and submitted by Geological Associates in November of 1975. A copy of that report is appended hereto, containing a detailed history of the reservoir together with a detailed description of the exploration and testing.

In essence, the investigations indicated that the reservoir structure is founded on bed rock overlain with highly weathered rock, interspersed with clay seams. These clay seams become saturated from seepage from the reservoir and provide slippage planes. It is the conclusion of the geological experts that the reservoir is in a marginal state of stability with respect to the subgrade.

POSSIBLE SOLUTIONS

In order to increase the factor of safety, six (6) possible solutions have been proposed for rehabilitation of the existing structure. In addition, a seventh alternate of abandoning the Eighth Avenue Reservoir and constructing a new reservoir at a different site was investigated, however, due to the lack of adjacent high elevation topography and due to the high cost of construction of a structure of comparable capacity (estimated $10 million dollars), this alternate was given low priority.

to injecting cement or chemical grouts into the subgrade, presumably to strengthen the foundation and reduce leakage. Both geological consultants, Geological Associates and Shannon and Wilson, question in their reports (copies appended) the feasibility of grouting. Neither consultant believes that grouting is a viable solution.

2. Waterproofing: Seepage through the walls and subgrade must be reduced to acceptable limits. Plasticity of the underlying clay seams is directly proportional to moisture content and therefore material reduction of seepage is paramount. Because of the fact that the inside walls of the reservoir have recently been regunited, it is probable that seepage is occurring through the bottom slabs and the joint between the walls and bottom. A flexible type lining would materially reduce seepage and would also overcome detrimental effects to water tightness caused by thermal movement within the structure.

The lining of each basin can be accomplished by utilization of the existing flexible, floating covers, by the removal of the floats and recementing the seams. The floats would then be reused for the installation of new covers. The estimated cost of providing linings and covers for both basins is $560,000.

3. Underpinning: The 1921 remedial work consisted of excavating narrow pits beneath a portion of the existing

wall and backfilling with concrete. Such underpinning work would of necessity need to be accomplished by hand without use of explosives and need to be staggered in order to maintain the integrity of the walls. Such work would be tedious and extremely expensive. We estimate the cost to be in excess of $6 million dollars.

4. Drilled Piers: To prevent slippage of the foundation material a possible solution would consist of a series of drilled piers, installed around the periphery of the structure, tied together at the ground surface with a post-tensioned ring. Investigation of this proposal indicates that the potential failure mode would be that of shear along one of the clay layers. Preliminary design of such a system indicates approximately 400 piers would be required with concrete cap at an estimated cost of about $1.34 million.

5. Enlarge Capacity and Underpin: This possible solution would entail excavation of the reservoir bottoms to an elevation of approximately 630. This would then allow "openface" mining of the foundation material under the walls for underpinning. This solution would have the advantage of doubling the capacity of the storage volume but would entail the disadvantage of requiring the entire reservoir to be removed from service and would be extremely more costly than the underpinning described under 3 above.

6. Tendons: This solution would consist of a system of circumferential beams, anchored to sound bedrock with inclined tension anchors. The system would be constructed

by installing beams or a ring around the outside circumference of the reservoir, drilling holes on 15 foot centers into sound bedrock, and inserting, anchoring, grouting, testing and locking off a suitable bar or stranded tendon at a precent of the design load. A sketch of this solution is shown in the appended Shannon and Wilson report and the appended Chester Engineers calculations. We have made a preliminary estimate of this scheme and it appears that this work could be accomplished for approximately $760,000.

MONITORING INSTRUMENTATION

Investigations to date, combined with prior history, indicate that the condition of the foundation is of marginal stability. It is proposed that a system of instrumentation be installed that would both assist in construction of any permanent repairs and would also provide an early warning of small detectable movements. The system would consist of 18 instruments spaced at about 90-foot centers. Each instrument would include 1/4-inch tubing or rod encased in 1/2-inch PVC tubing, all installed in a 2-inch diameter hole drilled into sound rock on a 30 degree angle. One end of the tubing or rod would be anchored into the rock, the other end attached to a linear potentiometer sensor. All sensors would be connected by underground cable to a central control terminal which would continuously scan and record outputs and sound an alarm if preset limits were exceeded. It is estimated that the cost of this equipment would approximate $26,000 and the cost of drilling and installation would approximate $7000 or an

estimated total of $33,000. A more detailed description of this instrumentation is contained in the appended Shannon and Wilson report.

MISCELLANEOUS IMPROVEMENTS

In addition to improving the structural integrity of the reservoir with the aforediscussed tendon system and waterproof lining, certain other miscellaneous improvements should also be made at this facility.

It is recommended that a concrete slab be constructed around the outer periphery of the reservoir, extending from the proposed tied-back ring to the existing drive. The purpose of this pavement slab would be to divert water away from the exterior of the structure and thus away from the reservoir wall foundation.

Several large valves and a sluice gate have become inoperable since first installed in 1889. These valves and gates are vital factors in the proper operation of the reservoir. In addition, it is recommended that an attitude valve be installed in the influent line which will provide more satisfactory operation in filling the reservoir and in diverting flow to other reservoirs on the general service pressure system. Estimated costs for this hydraulic work are quite high due to the absolute necessity of installing valves and tapping the lines while the system and reservoir remains in service to provide fire protection to the central business district.

Other miscellaneous work that needs to be accomplished at the reservoir to enhance its operation, are: the rewiring of

the gate house and reservoir lighting system; provision of adequate toilet facilities for operating personnel; and rehabilitation of the interior of the gate house structure. The estimated cost of construction of the recommended miscellaneous improvements is $350,000.

RECOMMENDATIONS

It is the consensus of opinion of Mr. R. T. Throckmorten, Jr., Geological Associates, Mr. Rudy J. Dietrich, Shannon and Wilson, Inc., Mr. William I. Gardner, Consulting Engineering Geologist and The Chester Engineers that the most feasible means of increasing the factor of safety against failure of the subgrade would consist of the following:

1. Immediately install the recommended extensometer system both prior to refilling the west basin and prior to any permanent repair work.
2. Convert existing flexible type covers to water tight liners and install new flexible, floating covers.
3. Construct the tendon or tied ring system as described under paragraph 6, anchoring the structure to sound bedrock.
4. Miscellaneous improvements including new and replacement valving and controls, waterproofing slab, rehabilitation of gate house, rest room facilities and rewiring.

The present total estimated construction cost, as shown on the following summary sheet, is $1,703,000. Adding approximately fifteen percent (± 15%) to cover engineering, administration,

inspection and construction contingencies would equate to a total estimated project cost of $1,960,000.

Respectfully submitted,

THE CHESTER ENGINEERS

T. A. Fithian, P. E.

SUMMARY

ESTIMATE OF COSTS

EIGHTH AVENUE SOUTH RESERVOIR

1.	Tied-back Ring & Tendon System		$ 760,000
2.	Monitoring Instrumentation		33,000
3.	Lining and New Cover		560,000
4.	Miscellaneous Improvements		
	a. Concrete Waterproofing Slab	$90,000	
	b. Rehabilitation Interior Valve Control Building	7,000	
	c. Replacement of West Basin Influent Gate	15,000	
	d. Effluent 36" Insert Gate Valve & Vault	60,000	
	e. Influent 36" Insert Gate Valve, 2-30" Tapping Sleeves & Valves, 30" Altitude Valve and Vaults	160,000	
	f. New Restroom Facilities	3,000	
	g. Rewiring of Gate House & Reservoir Lights	15,000	
			350,000
	Sub-total		$1,703,000
	Contingencies & Engineering(±15%)		257,000
	TOTAL ESTIMATED PROJECT COST		$1,960,000

APPENDIX A

Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee

November 25, 1975

VOLUME I	-	Text
VOLUME II	-	Appendices
VOLUME III	-	Drawings (See separate envelope)

Supplemental Report (February 20, 1976)

Geological Associates, Inc.

APPENDIX D

Estimates and Sketches

Tied-back Ring Beam & Drilled Piers

February 1976

The Chester Engineers

TIED-BACK RING BEAM

A circumferential ring beam has been designed around the base of the reservoir and this beam will be anchored to sound rock with inclined tension anchors. This scheme is illustrated in Figure 4 and Figure 5 of the Shannon and Wilson, Inc. Stability Review of January, 1976.

Their report states that stability can be attained by increasing the factor of safety against horizontal sliding by 0.5, thus providing an additional horizontal resisting capacity equal to 50 percent of the existing horizontal hydrostatic thrust, which would be equal to 15 kips per ft. It is their opinion that with this method we would not have to anchor the reconstructed or underpinned sections of the wall. Geologic Associates concurs with this conclusion.

We have designed a ring girder which is detailed on accompanying sketches #1 and #2. The elevations of this girder vary and are based on the base elevations of the wall. We have enclosed one detail of the girth beam which shows the typical size and reinforcing.

The basis of design for the beams are:

A) 15'-0" anchor spacing

B) Compressive stress between the beam and masonry wall 1875 p.s.f.

C) Compressive stress on bottom of girth beam 3700 p.s.f.

D) No girth beam required at the wall sections where the 1914 and 1921 reconstruction took place

E) 225^k per anchor loading lateral force

F) Required anchor force: $\frac{225 \text{ kips}}{\cos 30^\circ}$ = 260 kip anchor force

We have prepared a cost estimate on the cost of the tie back system.

1. No. of tendons required: 108
   Length of tendons vary from 41' to 83' with 15' min embedment into sound rock
   Total length of tendons equal 6800'
   The cost of tendons: 6800'x $20/ft $136,000

Cost estimate of tie back system (continued)

	Subtotal from previous page		$136,000
2.	Concrete Girth beam 2100 c.y. x $175		367,500
3.	Reinforcing 65 tons x $800		52,000
4.	Excavation 6200 x $6 c.y.		37,200
5.	Backfill 4100 c.y. x $2 c.y.		8,200
		Subtotal	$600,900
	Contractor's Overhead = 15% Contractor's Profit - 6%	Subtotal	$126,189
	General Requirements 4.5% Total Construction Cost 4.5% ($600,900)		$ 27,040
		TOTAL	$754,129

The second method that we investigated was the drilled piers with a nominal cap beam.

The stability analysis performed by Geologic Associates, Inc., is predicated on a potential mechanism of failure along a surface approximated by a series of planes - in this case nearly horizontal clay seams. Because of this potential type of failure, the primary mode of resistance of a system of piers would appear to be in shear rather than in cantilever bending.

For our cost estimate, we have assumed the thin failure plane and have assumed the horizontal pressure to be distributed uniformly above this plane to the drilled piers.

Below the failure plane we have assumed the drilled piers will resist the loading by passive pressure against the piers. The details are shown on the enclosed sketch.

The drilled piers are closely spaced together and should have the same effect as a continuous wall.

A cost estimate for the installation of the drilled piers is shown on the following page.

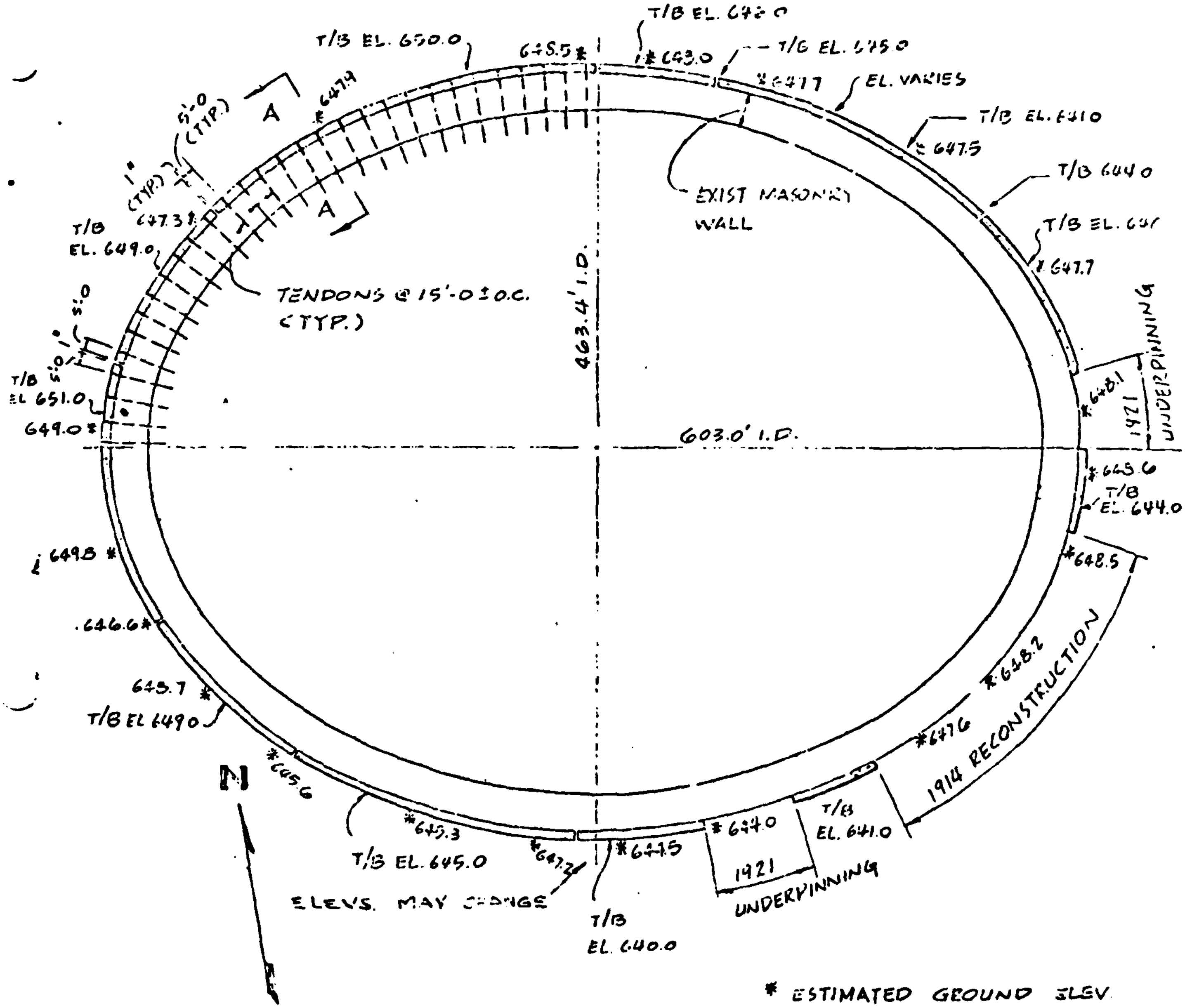
The cost estimate for the drilled pier system will be as follows:

1.	271-27' drilled piers = 7,317	
	68-18' drilled piers = 12,241	
	(WF 30 x 132) 132 x 8541' = 564 ton	
	72-20' drilled piers = 1,440	
	(WF 24 x 76) 76 x 1440 = 55 ton	
	Total for structural steel sections used in the drilled piers (619 tons) ($800) =	$ 495,200
	Total drilled piers (9981 lin.ft.)($15) =	149,715
2.	Concrete	
	Concrete cap 3' x 2' (408 c.y.) ($175)	71,400
3.	Reinforcing (12 tons)($800)	9,600
4.	Concrete around piers (1814 c.y.)($75)	136,050
5.	Excavation 6200 c.y. x $6/c.y.	37,200
6.	Backfill 5792 c.y. x $2/c.y.	$ 11,584
	Subtotal	$ 911,244
	Contingencies (10%),Contractors OH&P (15%)	$ 227,811
	TOTAL	$1,339,055

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

PLAN OF CONC. GIRTH BEAMS   SHEET NO.

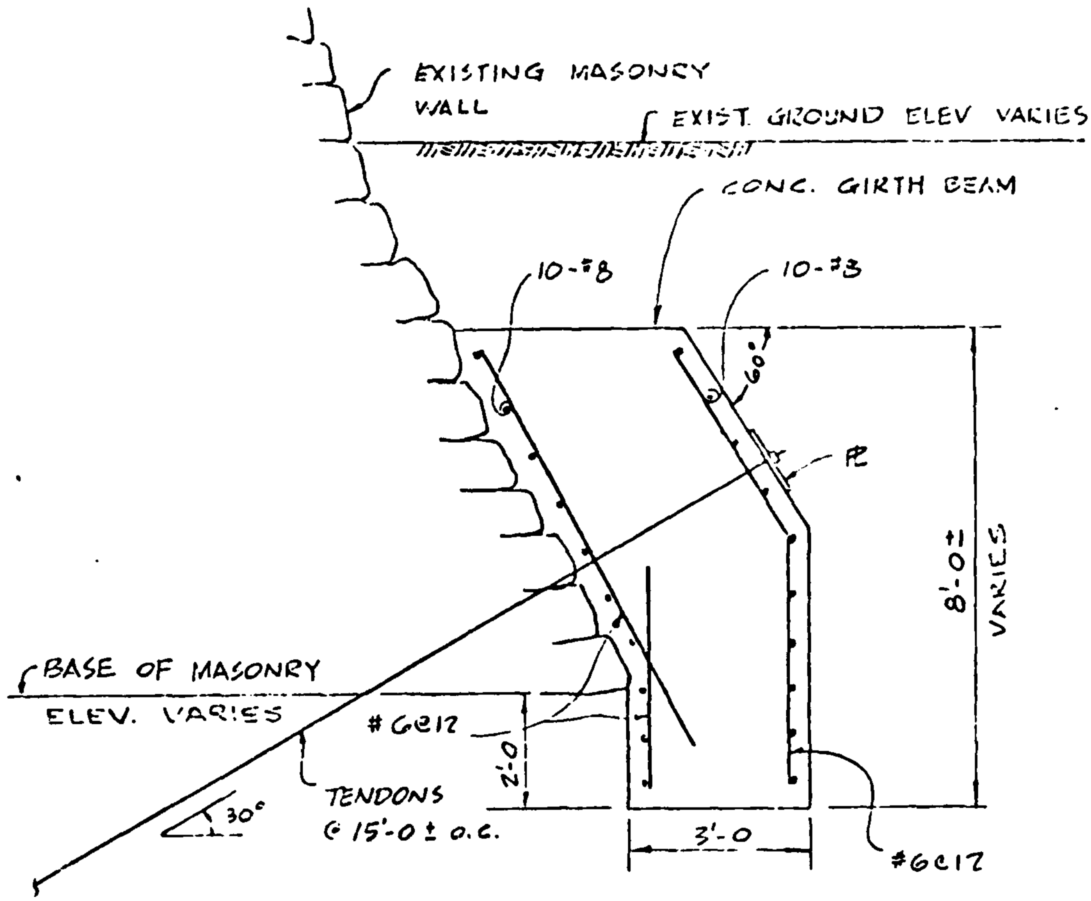
COMPUTED BY   CHECKED BY   DATE



PLAN
1" = 100'

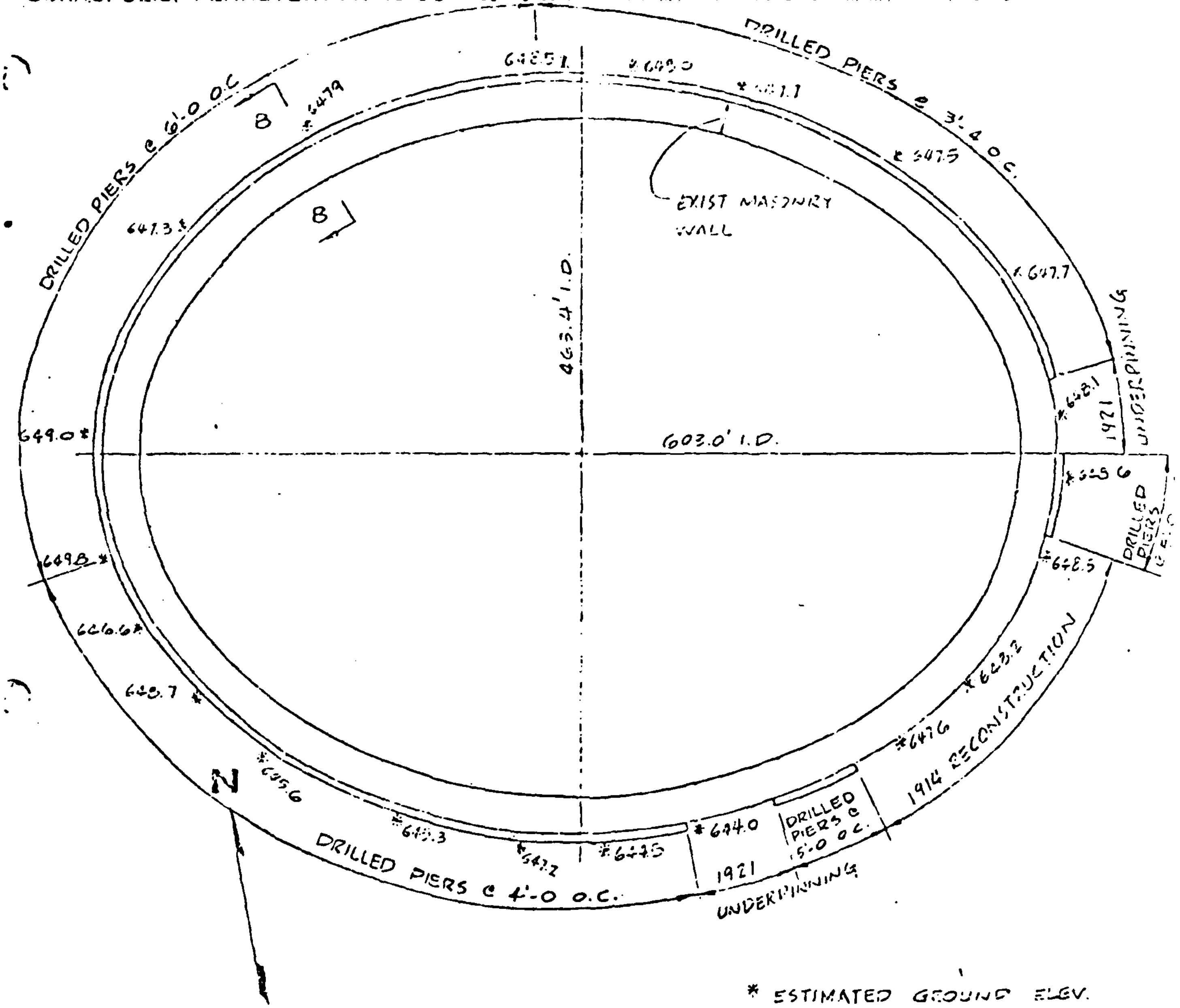
THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SHEET NO.
COMPUTED BY .... CHECKED BY .... DATE



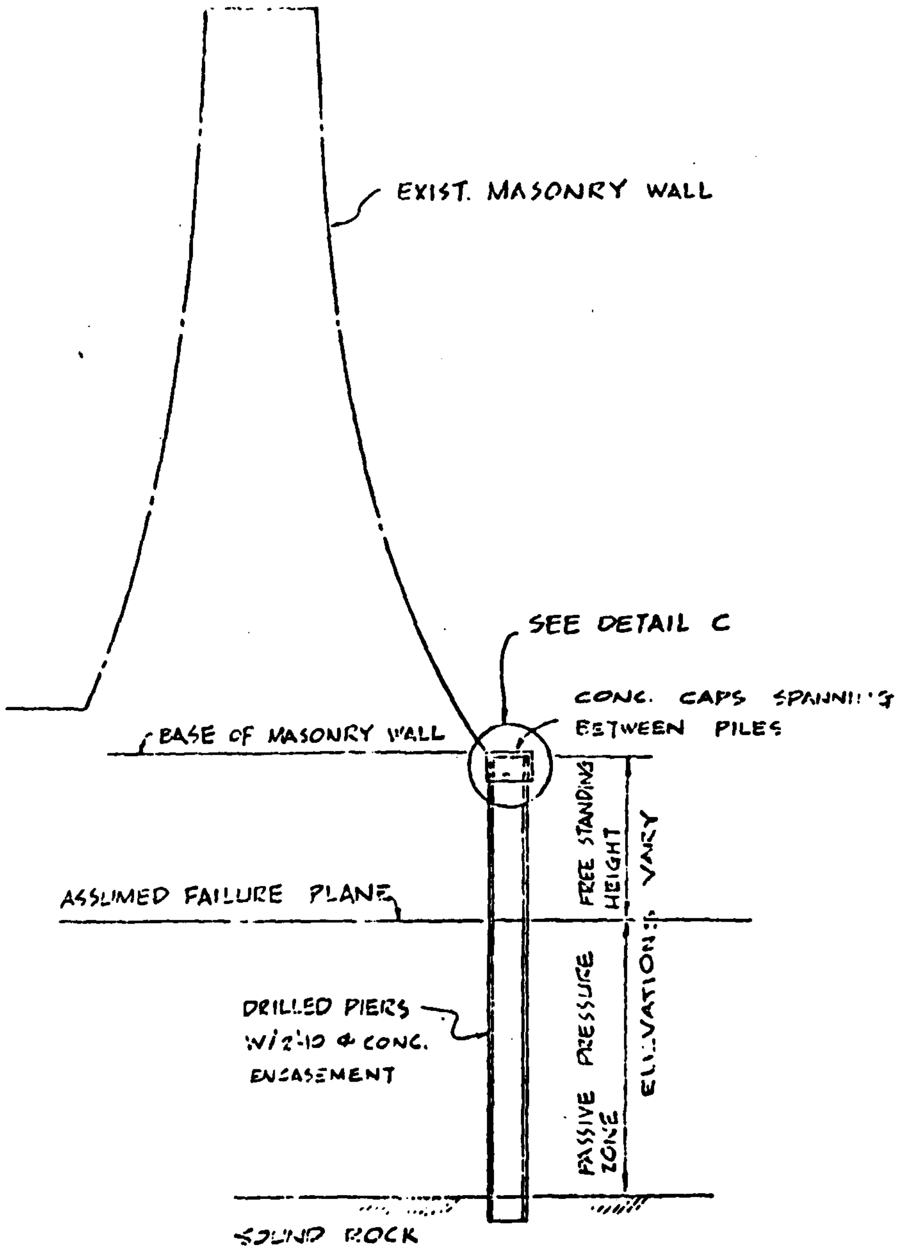
SECTION A
1" = 3'-0

845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108 COMPUTED BY CHECKED BY DATE



PLAN

1" = 100'



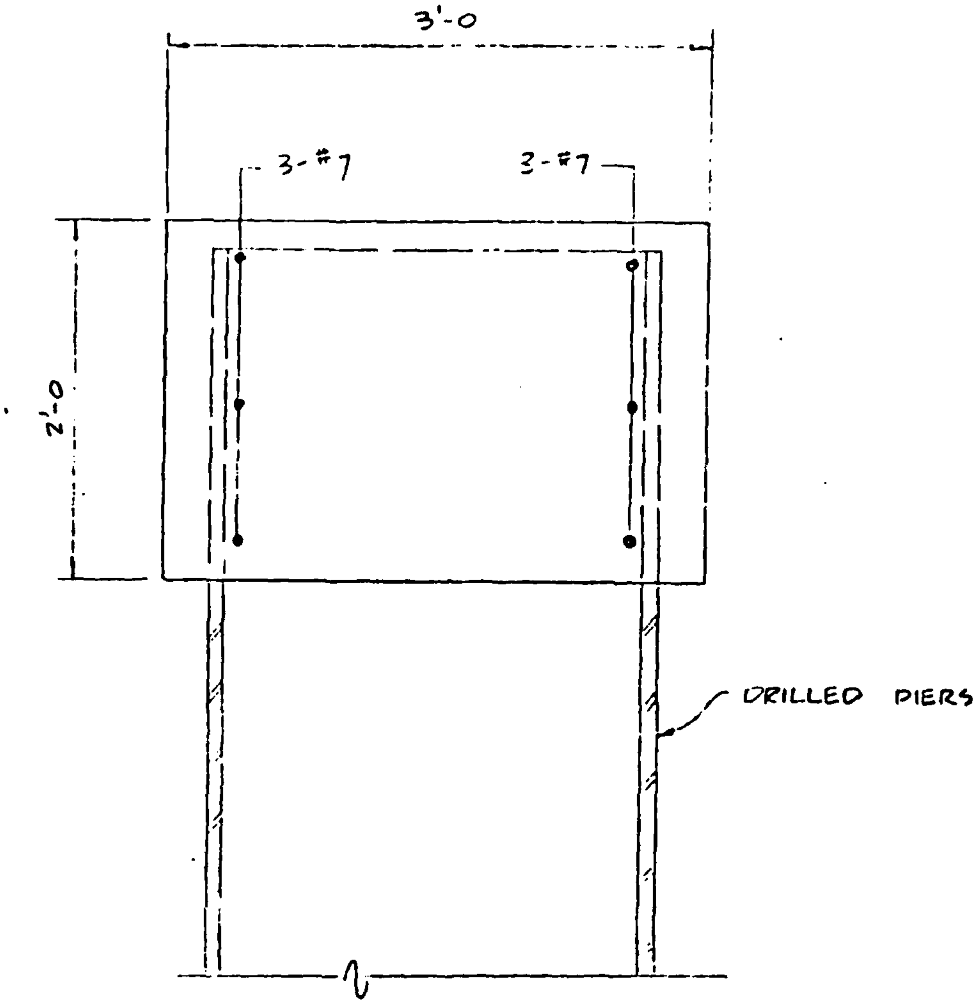
SECTION B
NOT TO SCALE

ASSUMPTION:

THE HORIZONTAL PRESSURE IS UNIFORMLY DISTRIBUTED ALONG THE ASSUMED FREE-STANDING HEIGHT.

MAXIMUM PASSIVE PRESSURE = 550 [illegible]

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108 COMPUTED BY .... CHECKED BY .... DATE SHEET NO



DETAIL C

1" = 1'-0

Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

Volume I - Text

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

November 25, 1975

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee 37201

Attention: Mr. K. R. Harrington

Gentlemen:

Re: Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

With reference to the above site, we have completed the second phase of the subsurface exploration, have analyzed all of the data gathered to date and present the latter and our conclusions herewith. Further, we have installed fourteen inclinometers in a pattern surrounding the reservoir which will permit us to monitor movements within the soil and rock mass comprising the subfoundation. We have also placed four tiltmeters on the top of the reservoir wall at its intersection with the axes. As we have pointed out, these monitoring systems will serve several purposes in addition to monitoring the stability of the structure. For example, we expect that the data will be of value in determining the scope and to some extent the design of any remedial work undertaken.

BRANCHES: P. O. BOX 9278 KNOXVILLE, TN. 37920 615— 573-7383
2711 FT. CAMPBELL BLVD. HOPKINSVILLE, KY. 42240 502—886-C721
P. O. BOX 988 KINGSPORT, TN. 615-246-4491

It seems obvious to us that the first order of business is to decide what are reasonable limits to the projected life span of the structure, especially under various conditions of repair and operations. Subsequently, the various suggestions made here, and those which may be developed, can be considered in their proper order and assessed as to their feasibility.

In the interim, we will continue to monitor the structure and will submit supplemental reports as facts and interpretations of them are available, as they may well affect the continuing operation of the structure.

In conclusion, we are available to discuss the data and our recommendations at your convenience.

Respectfully submitted,

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton, Jr., P.E.

RTT/mr

Copy: Mr. Ted Fithian

Enclosures: Text
Drawings, 7
Appendices, 3

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

SUBSURFACE INVESTIGATION
EIGHTH AVENUE RESERVOIR
NASHVILLE, TENNESSEE
PROJECT NO. 75-090

BACKGROUND OF THIS STUDY.

In the latter part of 1974, cleaning of the basins - for the first time in about forty years - was completed, along with some other construction efforts which included guniting the inside surface of the walls. The west basin was restored to operation during March 1975. Later in the Spring, seepage through the lower courses of masonry was noted along the north-northwest arc of the west basin. Although this area had displayed evidence of seepage for some years, the volume of "new" leakage exceeded any noted previously.

Initial observations seemed to indicate some diurnal influence in that flows increased during the afternoon hours when the sun's rays bore most directly on portions of the wall. Flows decreased or ceased altogether during the evening and morning hours.

On May 22, 1975, the writer first inspected the site in the company of Department engineers and consultants. Subsequently, we were directed to begin an exploratory program directed toward determining subsurface conditions adjacent to the visibly affected segment of the reservoir wall.

Data obtained during the first phase of exploration, which consisted of eleven core drill holes, indicated that the engineering qualities of the subsurface materials are very poor, to the extent that the subject arc of the wall undoubtedly exists in a precarious state of stability. Owing to the fact that continuing seepage of water into the subgrade could only further reduce the integrity of the foundation system, the west basin was drained - and remains so. Our initial study also hinted at the fact that perhaps much of the remainder of the reservoir's substructure rested on potentially unstable materials.

At the conclusion of a conference held July 30th, at which time the data then in hand were presented in some detail, we were directed to propose, and subsequently perform, a program of exploration and analysis which would define subsurface conditions adjacent to the entire perimeter of the structure. At that time we also proposed installing the monitoring system described in the transmittal letter for this report.

This report includes all of the data accumulated since last May, as well as our considered opinions based on the facts as we understand them.

HISTORY.

More than usual, a knowledge of the history of the construction and operation of this structure is essential to an understanding of its present status and as a means of inferring subsurface conditions which cannot be firmly established by exploration.

At the outset, we remark that a pursuit of the details of original construction, and reconstruction, has been largely a frustrating experience in that large gaps exist in the record. However, as soon as possible, we intend to further research the history of the engineering of this structure. What follows is a recapitulation of the operational history derived from the data we now have.

Construction of the Eighth Avenue South Reservoir on Kirkpatrick Hill began in August 1887, and was completed two years later. For many years it was referred to as the "Kirkpatrick Hill Reservoir", or more simply as, "The Reservoir."

The reservoir is elliptical in plan; the minor axis is 463.4 feet and the major axis is 603 feet in length. A wall along the minor axis divides the reservoir into two basins, each holding in excess of

25,000,000 gallons. The reservoir is oriented with the minor axis at about N16°E. The dividing wall was required initially to permit use of the west section as a raw water coagulation and settling basin; the east basin was used for storage of "treated" water.

We have been unable to determine who designed the structure, but some old drawings are available which show that a stability analysis was made for a typical wall section under the influence of the imposed hydraulic forces. The structure is of the gravity type in that the hydraulic forces are resisted solely by the mass of the wall. The perimeter and dividing walls were constructed of cut stone masonry facing with a stone rubble fill between. The interstices of the rubble were filled with concrete to produce a mass often referred to as "cyclopean concrete". The walls are battered on an arc and have differing inside and outside radii. Nominally, the walls are eight feet wide at the top (elevation 676.5), while at the basin floor (elevation 642.75) the walls are nearly 23 feet in width. Parenthetically, at some locations we have determined, and others previously reported, that the wall is 36 feet or more in width at its contact with the subgrade. (See Appendix B).

Construction was initiated in 1887 by the firm of Whitsitt and Adams and completed at a cost of nearly $365,000. We have been unable to locate any records of construction; however, we are quite sure (based on logic and lithology) that the stone masonry was obtained from the quarry located on a hill within view of the reservoir to the north near Archer Street and 10th Avenue, South. (The quarry is no longer in operation and has been filled.) We presume that the stone was supplied on a "low bid" basis as it certainly was not selected for either its durability, or ease of quarrying, for that matter. The stone is shaly, thin to medium bedded, occasionally nodular, poorly resistant to weathering, and certainly not very handsome. The quarrymen apparently could not constantly supply blocks of uniform dimensions, consequently, there is no uniformity to the number of masonry courses from place to place. Obviously, the stone masons had to make-do with what was delivered. The poor resistance of the stone to weathering, especially under the action of smoke acids, frost pry, and wetting and drying, has caused the faces of the masonry blocks to retreat as much as six inches in some places. Also, the intervening mortar has, understandably, weathered differentially. While this differential weathering is unsightly, it has had essentially no effect on the existing structural integrity of the wall. Nonetheless, before too long it will be necessary to consider methods of arresting the continued surface degradation of the stone by weathering processes.

Based on the subsurface exploration, we have determined that the present configuration of the hill bears little resemblence to the original topography. The crest of the hill must have been smaller in area and somewhat higher in elevation. The soil and weathered rock removed during grading for the structure, as well as the portion of the wall destroyed by the 1912 failure, have been used as uncontrolled fill to construct the slopes and berms which now comprise the grounds. A perusal of Profiles B and F, as well as logs of Holes 30 and 35, will give some indication as to the amount of fill which has been added from place to place.

Another indication that perhaps the original construction was less than first-class is the fact that numerous historical references are found alluding to the fact that the reservoir leaked from the time it was placed in operation. The Mayor's Annual Message for 1912 notes that the failure which occurred that year was, "directly due to faulty construction . . . . . many years ago." We suspect that this was a fact and not entirely due to a need to pass the political buck! Further, the 1912 Report of Waterworks Department states, "There has been considerable leakage from the reservoir ever since it was built." At any rate, prior to 1912, the city struggled along for 23 years with

a relatively new structure which leaked, and apparently with neither the time nor the funds with which to take it out of service and repair it.

By "reading between the lines", we believe that the reservoir could not be emptied for some time prior to the 1912 failure owing to work in progress on the main sewer into which it drained. The latter work was presumably being hurried along not because of alarm at the volume of leakage, but because the basins were badly in need of cleaning of sediment.

A few minutes after midnight on November 5, 1912, a segment of the east basin wall approximately 200 feet in length was displaced and a flood of water (the basin was essentially full) roared downhill toward Argyle Street and Eighth Avenue. Old photographs which we have had copied and enlarged permit considerable insight into the mechanism of the failure, as well as attest to the force of the wall of water which cascaded down the hill. Photographs show that the force of the flow was sufficient to sweep away the fill placed on the hillside and scour the natural ground to bedrock.

A tribute to the strength of the reservoir wall is the fact that a section about 100 feet in length remained intact as it was displaced outward and downhill.

The affected segment consisted of most of the southeast quadrant of the east basin. (See Sheet A). We have not found any engineering analysis of the failure nor any records pertaining to the details of reconstruction; however, we have reason to believe that a Mr. Rudolph Hering, "of New York City", may have served as a consultant to the city during the repairs. He was involved in a further study of the west basin in 1914, and by inference we assume he had been working for the city prior to that time.

Subsequent to the failure, we believe some time passed before reconstruction was undertaken while discussions were held regarding abandoning the site in favor of a new location. At any rate, the reservoir was ultimately repaired at a cost of about $100,000. A discussion of the visible extent and nature of the repairs appears further in our report. However, we mention here that during the reconstruction, in addition to rebuilding the wall, a perimeter french drain system was installed adjacent to and inside of the east basin wall and was drained to the

exterior at a point near the northern extremity of the reconstructed section. The east basin was placed in service again in 1914. As soon as the latter was restored to service, the west basin, which had been operated at a reduced head in the interim, was drained and cleaned of sediment by prisoners working around the clock Mr. Hering apparently had recommended some extensive repairs to the west basin which were subsequently deleted from the program after he re-examined the basin. Finances may well have influenced the decisions, because the cost of the repairs to the west basin were reduced from an estimated $45,000 to $27,000.*

Owing to the fact that numerous allusions were made in the above report to "water proofing", we presume that the west basin was leaking nearly as badly as the east basin had leaked.

Nonetheless, during 1914, six test pits were dug, "around the foot of the west basin for the purpose of finding leaks and also for examining the foundation for weak places."* At one location, "a very heavy buttress of concrete was built,"* to cover a weak place. We have not

* Annual Report, Waterworks Department, 1914

encountered this "buttress" during the recent exploration, unless it is possibly the concrete slab found during excavation of Test Pit No. 1.

As a further adjunct to the 1914 reconstruction and repairs, the interior of the walls was gunited and the floor of the basins was treated with asphalt and overlaid with concrete. Most of the $27,000 was probably spent on this effort. It seems obvious that the concensus was concerned primarily with preventing leakage from the structure and little consideration was given to conditions in the subgrade, except for the fact that they could deteriorate further if subjected to flowing water.

Finally, in 1914, regrading and improving of the reservoir grounds, etc., were completed to essentially the existing configuration.

Apparently, a close watch was kept on the reservoir after the 1912 disaster, and when distress again was noted in June 1920, forces were quickly mobilized to analyze the situation.

Mr. J. N. Chester, Pittsburgh, Pennsylvania, was engaged as a consultant to head an engineering committee composed of Messrs McDonald, Simpson and Brown. When the east basin was again removed from service, inspection

of the interior revealed a horizontal crack in the masonry wall about 26 feet below the top (6 - 7 feet above the basin floor) extending approximately 120 feet to the north from the east end of the major axis. From the committee's description of the visible distress, we guess that the base of the wall had shifted slightly downward and outward. Only minor vertical cracking in the lower masonry courses was noted on the exterior. This distress was cause for considerable alarm as it was essentially adjacent to, and north of, the segment which failed in 1912. (See Supplemental Plan, Sheet A).

The exploration conducted for this study was rather extensive and is much better documented than any previous. At least 14 exploratory holes were cored inside and outside the basin, including three drilled from the top of the wall through the affected segment. At least one hole (No. 9) was angled at 45°, and a horizontal hole was drilled under the wall from a test pit. Regarding the test pits, at least 8 were excavated in order to view subfoundation conditions. During this work the committee came to believe that a "fault" found in Test Pit No. 8 extended under the east basin along a line which projected its exit just east of the south minor axis. Three test pits were excavated in

this latter area, which lies just to the south and west of the 1914 reconstruction. We emphasize that the area affected and treated in 1920-21 essentially bracketed the 1912 failure area. (See Sheet A).

The core drilling in 1920 was under the direction of Captain George Reyer, Superintendent of Water Works, and the logs, although sketchy, are of value in that they reveal poor subsurface conditions, such as cavities, loss and migration of drill water and dye from place to place, etc. We presume that the drilling was done by skilled journeymen, yet the core recovery averaged less than 68%. Even allowing for the fact that drilling and coring equipment at that time were far less sophisticated, and core recovery would be expected to be less than with present tools, all of the cored holes indicated that a deeply weathered bedrock condition existed. The engineering committee and Mr. Chester described the presence of faults, enlarged high-angled fractures ("cutters") and softened shale layers in the core and test pits. They also noted that reservoir leakage along these paths "in years gone by", reduced the competency of the subgrade so that it, "possesses neither the weight-bearing power nor the resistance to sliding of the original . . . . ". Although the implication of these

data and statements were obvious, the committee recommended that only about 50% of the affected 120 feet of wall be underpinned. Ultimately two-thirds of the segment was underpinned. We point out that perhaps one reason they were quite restrictive in their recommendations was because sediment and other factors reduced storage in the west basin to about eight million gallons - at that time, a half-day supply. Immediately after New Year's, 1921, work began on the underpinning under the direction of A. W. Reppermund of The Chester Engineers. Labor and equipment were hired and supplied locally. Work continued without interruption until finished during the summer of 1921. During the underpinning, another french drain was constructed which was designed to train water seeping from the basin and collecting along the "fault line" to a manhole located in Test Pit No. 8. (See Sheet A). The subsurface details of all of this work are described further in our report.

As a part of the 1921 work, the interior walls of the entire reservoir were scaled and a new coating of gunite applied. Further, another concrete floor was laid over the old. As in 1914, guniting of the dividing wall was very difficult because of the numerous leaks issuing from the face - even under a reduced head in the neighboring basin.

As mentioned previously, no further major repairs or alterations were made to the structure until those recently completed.

PRESENT EXPLORATION AND TESTING.

The subsurface exploration consists of thirty-eight cored holes and the locations of these holes and the profile limits are shown on the Plan, Sheet 1. Subsurface relationships are depicted on the profiles and logs of the holes on Sheet 2 through 6. Sheet A is a Plan on which various historical data are shown as adapted from texts and drawings by others, as well as certain information developed during our investigation. Three appendices contain supplemental, pertinent information.

In all instances, the overburden was washbored and the bedrock cored size 'N' (2-1/8") or 'B' (1-5/8"). Most of the coring was done using high-recovery diamond drilling tools. Some of the initial coring was performed with a sophisticated split-barrel system not in common usage. Three of the cored holes were inclined at 45° in order to provide data not obtainable from the verical holes - which constituted the remainder of the exploration. Core recovery during the current exploration has averaged 99.5%, excluding cavities. Core recovery including cavities

has averaged 96.0%. The former percentage is always considered an index to the quality of the drilling operations, while the latter is a rough guide to the severity of bedrock weathering. The high percentage of recovery recorded during the current exploration including the non-rock (cavity fillings, clay seams, etc.) is an indication of the quality of the core recovery equipment utilized, as well as the skill of the drillers. As we have pointed out previously, it was essential that the non-rock portions of the weathered bedrock subgrade be recovered in order to permit a reliable assessment of conditions in the subfoundation.

Pipe was permanently cemented in several of the holes (1, 2, 3, 4, 6, 7) in the area around the northwest quadrant of the west basin in order that the static water levels could be observed in that sector. Continuing water level observations can also be made in Hole 20 near the east major axis. Representative water levels have been noted to the left of the drill hole logs; they often indicate the average of numerous readings spread over a considerable time interval.

Fourteen of the exploratory holes, ten adjacent to the reservoir wall and four outlying, were reamed and adapted for the installation of

special plastic casing required for the inclinometer monitoring system. These blue pipes are readily visible and must be protected from damage by vandalism, grass mowing, etc. The four ceramic tilt plates were grouted to the walkway on top of the basin wall at the quarter points. These, too, must not be disturbed. The holes prepared for instrumentation are being regularly "read" with a Digitilt Sensor biaxial accelerometer as manufactured by the Slope Indicator Company, Seattle, Washington. These data are being assimilated and analyzed by computer as they are gathered. The records compiled thus far have been utilized in preparing this report.

Further exploration consisted of excavating four test pits adjacent to the northwest arc; as shown on the Plan, Sheet 1 and on Sheet 3 as logs. These pits were backfilled with earth or lean concrete subsequent to inspection and photographing.

Representative portions of the clay seams recovered in the core were subjected to laboratory tests which included natural moisture content, particle size distribution, and Atterberg limits. These data were essential to an initial stability analysis performed using the Wedge

Analysis Method. The analysis and test results are attached as Appendix A. Bedrock cores are on file at our Franklin, Tennessee office.

SITE GEOLOGY.

Nashville is located on the west flank of a large structural bedrock arch known as the Nashville Dome. Therefore, in general, the bedrock strata in Nashville have a gentle areal dip to the northwest. However, superimposed on the general dip are many, many local variations and reversals to the regional dip. In addition, numerous bedrock faults have disrupted the continuity of strata from place to place. Most of the faults are of the normal, gravity, type and are presumed to have been inactive throughout historical times, and movement is not likely to occur along them if a seismic event occurs in the area. Parenthetically, Nashville is located within Seismic Risk Zone I; however, it is "sandwiched" between Zone II contours to the east and west. Assessing the stability of this structure under seismic loading is, at best, an exercise in empirical arithmetic.

Tension fracturing (jointing) of the bedrock is generally of more significance to the acceleration of bedrock weathering than faulting.

because the former is omnipresent in the bedrock system. Jointing usually occurs in at least two sets (4 azimuths) and, depending on the frequency of spacing, has a profound effect on the depth and degree of bedrock degradation from place to place. Nearly all of the streams in this area, tributary as well as master, are adjusted to the fracture patterns as is attested to, for example, by the constant meandering of the Cumberland River.

Kirkpatrick Hill is an erosional remnant produced during the general inversion of relief occasioned by the river cutting across the dome. The strata immediately underlying the reservoir consist of shaly limestones of the Catheys Formation which is Ordovician in age. Portions of the foundation bedrock are degraded to relatively shallow depths, apparently where fracturing is modest. On the other hand, the juxtaposition of fairly intense fracturing and postulated gravity faulting under other portions of the reservoir have permitted degradation by percolating water and other weathering processes to depths considerably in excess of "normal" for these rock types. We cannot attest to what extent the percolation of water leaking from the reservoir over nearly a 90 year period has contributed to the bedrock weathering. However, we doubt that it has appreciably accelerated the long-term weathering processes. Nonetheless, percolating waters will certainly reduce the strength characteristics of the materials already reduced to clay.

A detailed geologic profile of the bedrock underlying the structure has been developed in the course of providing positive stratigraphic correlation from place to place. Appended to this report is a description of the bedrock members which identifies the letter designations shown on the left of the logs of the holes.

DETAILED DESCRIPTION OF STRUCTURAL AND SUBSURFACE CONDITIONS.

This portion of our report is best studied during a perusal of the pertinent geologic profiles. While all of the basic data and our interpretation of it are included on the drawings, this text can serve to illuminate and point out important details.

For discussion purposes we have divided the reservoir perimeter into sections based on subsurface conditions. (See Profile 'D-D')

General. The general dip of the strata under the reservoir is to the slightly north of west. Based on our interpretation of the bedrock structure, three gravity type faults cross the reservoir subgrade. They are roughly parallel and their postulated orientation is shown on the Plan, Sheet I. They are further identified by number on the profiles. We mention at this point

that we can find no evidence of faulting near the east major axis, as was postulated by the 1921 study. We believe that possibly some intense fracturing was incorrectly identified as a fault. On the other hand, we do postulate a fault (No. 2) near the south major axis extending through the 1921 underpinning. The rather complex bedrock conditions, coupled with the necessity for confining the exploration to the exterior of the structure, has necessitated construction of a simple model of the reservoir substructure. The model, constructed to the scale of the profiles, 1 inch equals 10 feet, permitted studying the substructure data in their proper elliptical configuration.

Northwest Section: (West Basin; vicinity of Hole 25 clockwise to Hole 12; ≈ 500 feet) This section of the wall was seeping the worst at the time this study was initiated and approximately brackets Hole 10. The area to the north-northwest of this section was explored in some detail during the first phase of exploration. Profiles 'A-A', 'B-B' and 'C-C' should be studied in conjunction with this segment of 'D-D'.

Surprisingly, on Profile 'D-D' for the most part, bedrock weathering extends to only modest depths, - on the order of less than 10 feet (elevation 635-640). Hole 11 is an exception which may possibly be attributed to its proximity to Fault No. 1, which incidentally seems to be the major fault crossing the site. The stratigraphic throw on Fault No. 1 is postulated to be eight feet at this intersection with the profile.

Based on the test pits and the holes, we believe that the foundation for the reservoir wall in this segment rests at about the level described and drawn as the "Diagrammatic Top Of Rock", which varies from elevation 640 - 646.

<u>Northeast Section:</u> (East Basin; vicinity Hole 12 to the north edge of the 1921 remedial work (Hole 20); ≈ 140 feet)

The bedrock under this reach of the wall is deeply weathered - possibly the most extreme at the site.

A perusal of the logs for Holes 34, 13, 26 and 14 will reveal that severe bedrock degradation extends to elevation 620, and below. Of further significance is the fact that during the

original construction the surficial materials must have been soft because the foundation contact is as low as elevation 635 (See Hole 26). It is possible that the wall is founded somewhat higher or lower from place to place, but we doubt that this variation exceeds 3 to 4 feet.

The north extremity of the interior crack discovered in 1920 must have been near Hole 14 (at profile distance 810). Further, observe the relationship of the stratigraphic depth of underpinning in 1921 (near Hole 20) to the base of weathering in Hole 14.

Apparently, during the original construction the batter on the walls was extended to the footing contact in all instances, as may be noted from the logs as well as the crossections of the wall presented in Appendix 'B'. These latter sketches are included because they are of use in studying remedial measures, as well as in stability calculations.

<u>Southeast Section:</u> (Sections underpinned in 1921 and 1914 reconstruction; ≈ 440 feet)

Initially, let us examine and discuss the segment replaced in 1914, the limits of which we have inferred and shown on this profile, and on supplemental Plan, Sheet A.

Based on the photographs taken after the failure, we conclude that a considerable portion of the weak, readily eroded, subgrade was piped and scoured from under the wall during an initial stage of the failure. When sufficient support was lost, the wall was displaced downward and outward under the weight of the water. It appears that the failure occurred as a typical wedge type. We also surmise that the structural failure was sudden because of the fashion in which some of the pieces were subsequently arranged. Based on the photos it looks as though the initial break occurred essentially along the trace of Fault No. 1. On this side of the reservoir the stratigraphic displacement of Fault No. 1 is calculated to be on the order of 13 feet, - the largest increment found by the exploration.

Based on the three holes (15, 22, 31) drilled along this reach of the wall, we surmise that most of the "new" foundation extends essentially through the weathered zone to sound, unweathered rock. An exception is noted in Hole 31, which lies to the west of the

fault. Because the test holes are located some few feet outside of the wall, prior to final design of remedial work, we will have to drill an inclined hole to determine the actual limits of "1914" excavation in this reach.

The 1914 foundation reconstruction consists of concrete containing fairly fine aggregate into which large chunks of rock were cast to provide bulk. This was a fairly common practice, and it looks as though some of the original stone masonry may have been used for this purpose. In general, the concrete appears to be in excellent condition. We presume that the concrete foundation was cast in a trench cut to "sound" rock and that the battered masonry walls were erected on it starting at elevations varying from place to place. This section will require further exploration prior to final design.

The above-ground reconstruction of the wall in 1914 followed the original curvature and batters. However, the masonry stone utilized has proved far more durable than that quarried from the original source. The stone used for reconstruction is a pinkish-gray, medium crystalline, massive limestone. We have no record

of its origin, but we suspect that it is Silurian in age and was probably quarried to the west of Nashville. Owing to the differences in color and lithology between the latter stone and the original, the fan-shaped reconstruction section is readily identifiable.

On the north side of the 1914 section there exists a reach of about sixty feet - extending to the east axis - which was not treated, but certainly should have been (See Hole 37). Presumably, at that time this bedrock looked adequate compared to that which had been removed. Further, it extended under a section of the wall not visibly affected at that time.

North of the east axis, portions of a segment fifty feet in length were underpinned in 1921. Here, too, the concrete is in acceptable condition and contains large slabs of rock cast into the concrete. Of this section, a block of rock about eight feet in width (under the wall) was not repaired. Mr. Reppermund's available notes do not state why this section was omitted.

Your attention is called to Profile 'E-E' on which the inclined Hole No. 21 is shown. Once again, the 1921 underpinned depth

correlates well with the stratigraphic level to which the 1914 reconstruction extended, and the wall is now in contact with sound, unweathered bedrock.

The other portion of the wall underpinned in 1921 begins sixty feet west of the 1914 zone and extends for about sixty feet toward the south minor axis. As shown, two portions, each about eight feet in width, were omitted from the underpinning program. Parenthetically, it is indeed unfortunate that not all of Mr. Reppermund's diary has been found, as it would probably provide a clue as to why these segments went untreated.

Nonetheless, the limits and elevations of the portions treated as shown on the profiles are based on an analyses of his notes and as-built drawings furnished by The Chester Engineers.

Fault No. 2 is postulated to cross the wall within this reach. It may well be that this rupture is the fault identified in 1921 and thought then to be an extension of a similar feature described as being encountered by underpinning at the northern location. We note again that we have been unable to identify any displacement due to faulting in the area adjacent to

Holes 20 and 21. Further, based on our analysis of all of the exploration, as well as experience with local geological conditions, we believe that all of the faulting is oriented as shown by our drawings, and that it is unlikely that any cross-faulting of significance exists. The postulated "fault" near the east major axis intersection was possibly a badly weathered and displaced tension fracture.

If conditions found in Hole 16 are fairly representative of the western reach between the 1914 and the 1921 remedial work, this section, sixty feet in length, should certainly have been underpinned. Note, however, that the base of the original stone masonry is at about elevation 635, and therefore, essentially "matches" the level of the 1914 rebuilding. We guess that in this area as well, deep bedrock weathering had necessitated more extensive excavation during the original construction. We estimate that a reach extending from Fault No. 1 to the vicinity of Hole 16 still has an average of 10 feet, to elevation 625, of badly weathered rock underlying the wall.

Nonetheless, this quadrant of the reservoir wall foundation is in relatively good condition with respect to resisting compressive forces, as well as in comparison with subsurface conditions under the remainder of the reservoir.

Southwest Section: (Extends from 70 feet east of south minor axis westward to vicinity of Hole 25; ≈ 450 feet, perimeter completed.) This section of the foundation bedrock is deeply weathered and constitutes the most complex geology on the site. Fault No. 3 enters and exits the reservoir subgrade in this segment. The rather narrow block of rock between it and Fault No. 2 (See Hole 33) constitutes a graben in that it has been dropped down with respect to the bordering strata.

The net result of these displacements, coupled with intensified fracturing, has been to enhance severe bedrock weathering. For example, the core from Hole 18, which transects Fault No. 3, is rather severely degraded to elevation 610, some twenty-five feet or more below the presumed existing foundation contact elevation. Based on inclined Hole 38, we suspect that much of this segment of the wall is founded at about elevation 635, or higher.

INCLINOMETER RECORDS.

The sensor being used to monitor movements within the subgrade is a biaxial instrument. The special casing installed in each hole is oriented so that movements normal and tangential to the wall can be observed. For convenience, the outlying holes were cased so that one axis is along a line normal to the reservoir wall. The special casings were cemented in the exploratory holes to depths sufficient to extend well into sound, unweathered rock in which no movement would be likely to occur.

For purposes of this installation, the 'A' axis readings are normal to the wall, and the 'B' axis records are tangential. The records show movement in the positive or negative (+ or -) direction. Negative displacement on the 'A' axis indicates movements toward the wall while positive readings show movement away from the wall. Movements in the 'B' axis are identified as minus (-) to the right and plus (+) to the left, as viewed facing the exterior of the wall in each instance.

The readings amassed to date are included as Appendix 'C'; we point out that they are currently in rough form and are for information purposes, only.

While the sum and import of the readings to date are incomplete and inconclusive, some trends and aspects are of interest. Contrary to what one might have expected, that is, movement normal to the wall, the slight movements indicated have been tangential. Further, the recorded 'B' axis displacements have generally been down-dip on the bedrock. There are some aspects to the current readings which imply that the west basin sub-foundation materials may be rebounding as a result of the lessened stresses imposed subsequent to draining.

As soon as a definitive pattern has emerged from the readings, we may recommend that the west basin be filled and monitored on a daily basis. We fully realize the importance of returning this half of the reservoir to service if it can be done safely.

With reference to the tilt plates on the reservoir rim, no observable movement has taken place in any of them.

THERMAL EFFECTS.

The coefficient of linear expansion for the masonry is estimated to be on the order of 0.00035 (for ^{o}F). Even allowing for the fact that nearly 1,800 feet of wall are thermally affected, the diurnal stress

buildup must be slight. Presuming that no more than half of the perimeter is being heated (while the other half may be cooling), and conservatively estimating a 50° F. temperature rise, we estimate the gross change in length may approach 1.6 to 1.9 inches (in 900 feet). Further, we suspect that the mortar may have a coefficient of expansion at least twice that of the stone. While the increments of movement being considered are small, we suspect that the exterior moves appreciably under the diurnal influence, in relation to the wetted interior which remains at an essentially constant temperature.

LEAKAGE FROM THE RESERVOIR.

While the aforementioned observable leakage in the northwest arc of wall was the initial reason for beginning this study, we have come to consider it a symptom of the problem, as well as a contributing factor, rather than the problem itself.

During our initial inspection of the reservoir and grounds last Spring, we observed a seep and persistent "wet spot" at the toe of the slope adjacent to the juncture of Alley No. 653 and Ninth Avenue. This area dried-up when the west basin was drained. Further, persons involved in sewer installation to the south during the Edgehill renewal program

reported having to contend with "unusual" quantities of groundwater. At the time, this condition was thought to be due to leakage from a swimming pool - now abandoned - located in Reservoir Park playground.

In addition, in Captain Reyer's October 1920 report he alludes to leakage by stating, "There is no lessening of flow in spring Southwest of reservoir since east basin was emptied." The inference here is that the observed spring was being supplied by leakage from the west basin. We have attempted to locate the area to which he referred to no avail.

In the light of the over-all information now in hand, we presume that the reservoir probably leaks to varying degrees around (and under) its entire circumference. Owing to the very size of the structure, as well as its mode of construction, we doubt that it is possible to keep it from leaking to some degree without installing an impermeable membrane over the entire wetted surface. Were it not for the continuing deleterious effect of even meager amounts of percolating water on the clay seams and other planes of weakness in the subfoundation, the present leakage would not be of as great concern.

We conclude that no real saturated level exists as a perched static water level under the structure because of the wide range in the observed water levels. We attribute this to the fact that openings (weaknesses) in the bedrock, such as bedding planes, high angled fracture and fault traces generally allow the water to escape downward without much lateral travel until the base of weathering is reached. These conclusions are based on the varying rates and degrees of recovery in the water levels recorded in the bore holes subsequent to bailing.

On the other hand, the segment of wall which displays seepage above the exterior ground level may well do so because the subgrade in that area is sufficiently tight (or saturated) as to not permit ready transport of the seepage downward and away from the site. It is quite possible to envision water traveling under the reservoir floor at shallow depths to the wall area and thence upward through the rubble interior until leakage and frictional loss balance the head. We have further evidence that, while the volume of water lost through this segment is not great, there is an appreciable head acting. When the west basin was drained, the static water level in the holes which had proved most responsive to minor fluctuations in the reservoir level immediately receded as much as four feet (See "a" and "e", before and after, levels noted on Profile 'A'-'A', Sheet 2).

We, therefore, conclude that essentially all of the subgrade for the reservoir may be subject to varying, but persistant, percolation of water from the basin, - and probably mostly from the floor area. Much of the residual clay in seams found in the bedrock is moderately to highly plastic and is, therefore, of low permeability. On the other hand, once wet the clay seams are not readily dried and returned to a more competent condition. The ramifications of these physical aspects are covered in some detail by the stability analysis included as Appendix 'A'.

RECOMMENDATIONS.

In the light of the data in hand, it is our considered opinion that the reservoir, at best, has always existed in a marginal state of stability with respect to the subgrade. The age of the structural components, coupled with continuing leakage, have served only to lessen the factor of safety against sliding failure under which its walls operate. Owing to the length of time during which the subgrade has had an opportunity to consolidate under the influence of the structural and fluid surcharge there is little danger of local bearing capacity failure, unless unusual piping occurs. However, the possibility for a sliding failure exists. This latter action would probably occur as a wedge with the affected

interval sliding en echelon along continuous clay seams within the weathered bedrock zone. Owing to the fact that considerable sums of money have recently been spent to gunite the walls and cover the basins, plus the fact that no other comparable site is available for construction of a similarly sized, new structure, we believe that extensive, thorough repairs, or reconstruction, are justified. Because of the marginal degree of safety now known to be inherent in continuing operations under existing conditions, we strongly urge you to proceed with design, funding and execution of whatever remedial work may be required.

POSSIBLE SOLUTIONS.

During our study we have given thought to many methods and combinations of systems for enhancing the integrity of the structure. The more we have studied the problem, the more complex it becomes, especially because of the weighting effect of time, cost, etc. We present the following comments as representative of our thinking at this time. We expect to refine our recommendations as data is amassed. The obvious advantages and disadvantages of each current suggestion are mentioned.

1. Pressure grouting coupled with continued monitoring. It is possible that a curtain consisting of holes inclined in two directions and pressure grouted with cement or mortar would prevent lateral seepage under the reservoir wall and would enhance the stability of the subgrade against sliding. Other than a water-proof lining, there is no practical way to prevent seepage from the basin affecting the materials directly beneath unless the basins are removed from service and a pattern of grouting installed in each floor. While the latter process may not be considered feasible, the curtain adjacent to the wall may be. As a minimum, we envision drilling at least a double row of grout holes around the perimeter. One row would be inclined under the wall and the other would be inclined tangent to the wall. In this fashion both horizontal and high-angled openings would be intercepted and treated. The use of chemical grouts or additives might be indicated.

The grouting treatment method would disturb routine operation of the reservoir very little, but it is certainly the least trustworthy in that grout can only be injected into existing voids, and we think it it impossible for the grout to displace any of the clay filling the

weathered seams. This approach does have merit if it can be demonstrated by the monitoring system that an appreciable reduction in seepage and a corresponding lowering of the level of saturation will produce a sufficient enhancement in the stability of the subgrade. (See Appendix A) We have not hydraulically pressure tested any of the drill holes because we did not want to forcibly pass water through the weathered seams unless the execution of a grouting program appeared to be a reasonable solution. Therefore, we can only make an educated guess as to the amount of drilling and grouting which might be required. We estimate that the grouting program as outlined above might cost nearly $500,000, based on current prices.

2. Waterproof lining of basins. Owing to the fact that a grout program might cost one-half million dollars, or more, we consider the lining of the basins as a reasonable alternative. Once again, if we can demonstrate by extended study and monitoring of the inclinometers that preventing seepage to the subgrade will improve its stability to an acceptable degree and thereby preclude a sliding failure, lining the reservoir must be considered. We prefer this idea instead of grouting because it is more likely to be nearly 100% effective in reducing seepage.

Further, because the solutions stated subsequently are likely to cost several times as much as 1 and 2, described above, it may be prudent to line the basins and monitor the performance of the reservoir while extended study and thought are devoted to an ultimate solution. We point out again that before too many years pass, the structure, as is, is sure to require extensive repairs. Although similarly designed and constructed facilities have been in use for hundreds of years, this one was not very well built, initially. During these deliberations a careful assessment must be made of the lifetime remaining for the reservoir.

3. Conventional underpinning of the wall in a fashion similar to that done in the past. This operation would involve excavating the weathered rock from under the wall on an alternating segment basis. The depths of underpinning would vary from less than ten to more than thirty feet, based on the exploration and refined by inspection during construction. If this method were selected, a modest amount of additional subsurface information would permit us to establish a reliable "proposed limit of excavation" line for detailed design and bidding purposes. The fact

that essentially 20-25% of the reservoir wall is already underpinned acceptably may make this solution attractive. We emphasize that owing to the fact that another 55 years have passed since the last underpinning, the wall may now be unable to safely span even eight feet unsupported. The actual parameters which could be utilized by an underpinning program would have to be determined by instrumentation during reconstruction. Also, it will not be possible to use explosives. Excavation by percussion drilling, barring, wedging, hydraulic fracturing, etc., are all very expensive modes of excavation, especially in restricted working room such as this. Further, by this method each basin will be out of service for a protracted period and, at some point, both basins would probably have to be emptied.

Because this is a "one-of-a-kind" sort of structure, and owing to the considerable depth of excavation which would be required, the ultimate feasibility of this suggestion should be discussed with one or more underpinning contractors who are very experienced in this field prior to more than tentative discussions by your engineers regarding its use.

4. <u>A system of drilled piers extending to, and into, sound bedrock, acting as cantilevered piers sustaining a post-tensioned "ring" retention system.</u> The various aspects, and adaptions of this, too,

must be explored by your structural engineers. But, this treatment will have to encircle the entire perimeter, including that already underpinned.

We also envision a considerable variation in diameter, length and spacing for the piers from place to place owing to the diversity of bedrock conditions. Presumably, only one half of the reservoir would have to be removed from service at any one time during this installation. We also recommend that one or more experienced drilled pier contractors be invited to inspect the site, view the data and provide us with their assessment of the projected work. We emphasize that it will be essential that the shafts be <u>drilled</u>, rather than excavated by blasting and mucking.

5. <u>Major reconstruction and enlarging capacity.</u> The most complex solution considered involves removing the entire reservoir from service (presuming that this is feasible after the Stones River plant is in full operation) and virtually reconstructing the reservoir on its present site.

The basin floor would be removed and a general excavation of the weathered bedrock, perhaps to elevation 630, completed. Further excavation under the walls to sound bedrock (where necessary) would be done by increments. The masonry walls could be carefully inspected, repaired and supported from below on concrete walls founded on sound bedrock. In this fashion the above-ground appearance of the reservoir would be retained, the structure could be made stable, and its capacity would be increased by something over 25,000,000 gallons. A reservoir floor level of 630, or less, would still be higher than other hills which have been considered as possible sites for a new or additional reservoir.

We will be glad to consider any other repair and reconstruction schemes which appear valid.

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton Jr.

R. T. Throckmorton, Jr., P.E.

RTT/mr

Enclosures: Drawings, 7
Appendices, 3

EIGHTH AVENUE RESERVOIR

GEOLOGIC COLUMN

AVERAGE THICKNESS IN FEET	MEMBER	DESCRIPTION
21 +	A	LIMESTONE, dense to fine to occasionally medium crystalline, gray, thin to medium bedded; with wavy laminae of dark-gray SHALE.
13.1	B	LIMESTONE, densely crystalline, gray; interbedded with alternating zones of numerous or occasional wavy bands and mottling of dark-gray SHALE.
2.8	C	LIMESTONE, densely crystalline to argillaceous, greenish-gray.
7.1	D	LIMESTONE, densely crystalline, gray, massive; with fairly regular bands and mottling of fossiliferous, dark-gray SHALE.
3.5	E	LIMESTONE, densely crystalline to argillaceous, greenish-gray. Persistant ± 0.5' black SHALE band at base.
2.4	F	LIMESTONE, finely crystalline, gray to bluish-gray, medium bedded; with bands and occasional laminae of dark-gray SHALE.
8.3	G	LIMESTONE, finely crystalline, gray; interbedded with LIMESTONE, medium to coarsely crystalline, gray; all having partings and occasional mottling of dark-gray SHALE. SHALE is occasionally very fossiliferous (coralline).
9.8	H	LIMESTONE, finely crystalline, gray, heavily mottled with fossiliferous, dark-gray SHALE.
3.7 +	J	LIMESTONE, finely crystalline, gray, thin-bedded, occasionally mottled with fossiliferous, dark-gray SHALE.
71.7 +	TOTAL	

Subsurface Investigation
Eighth Avenue Reservoir
Nashville, Tennessee
Project No. 75-090

Volume II - Appendices

APPENDIX A

STABILITY ANALYSIS

AND

PHYSICAL TEST DATA

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

EIGHTH AVENUE RESERVOIR STABILITY ANALYSIS

APPENDIX A

November 25, 1975
Project No. 75-090

A stability analysis of the foundation for the Reservoir was performed using the Wedge Method of Analysis. This method assumes that failure may occur in the subfoundation along a surface approximated by a series of planes - in this case nearly horizontal clay seams. Forces imposed by the structure and the surcharge of various stages of fluid head were resolved based on the available structural data and the geometry of the foundation. The configuration of the 1912 failure as recorded by photographs was considered in selecting the boundary for the active earth pressures in the light of detailed geologic data obtained from our subsurface exploration. Several potential failure surfaces were selected after studying the weaknesses in the subfoundation materials recovered by the core drilling program. Physical properties of the soil-rock system were estimated using basic laboratory test data.

Owing to the nature of the subfoundation materials, significant variations in the soil-rock properties occur within the zone adversely affected by the structural load. The variations which are most likely to affect the stability are lateral or planar features - namely, nearly horizontal, continuous layers of clay. The analysis shows that minor variations in the strength of the clay, as well as the position of the phreatic surface,

BRANCHES: P. O. BOX 9278 KNOXVILLE, TN. 37920 615—573-7383
2711 FT. CAMPBELL BLVD. HOPKINSVILLE, KY. 42240 502—886-0721
P. O. BOX 988 KINGSPORT, TN. 615-246-4491

have a substantial effect on the stability of the foundation and surrounding earth mass. For example, in some cases the margin of safety for stability is increased by from 20% to as much as 30% if the phreatic surface is lowered below the potential failure surface. Similarly, variations in the soil's shear strength on the order of 700 PSF can cause a 50% variation in the safety factor. Further, our analysis indicates that a 50% reduction in the Reservoir head would result in an increase of 40% in the stability safety factor.

Based on the stability analysis and data obtained from the inclinometers, we conclude that under the influence of the full reservoir surcharge, the existing stability margin of safety for critical segments of the wall is on the order 1.2. If seepage is not abated, under the influence of sustained full reservoir conditions, the phreatic surface within the underlying and adjacent soil system will continue to slowly increase the moisture content of the clays, thereby further reducing their shear strength and the factor of safety for stability. On the other hand, if the seepage can be prevented from affecting the soil system outside of the perimeter of the wall, the integrity of the soil system will slowly improve and the margin of safety for stability will correspondingly increase. However, because of the low coefficient of permeability of the clay (on the order of 10^{-7} cm/sec.) moisture reduction and associated shear strength increase

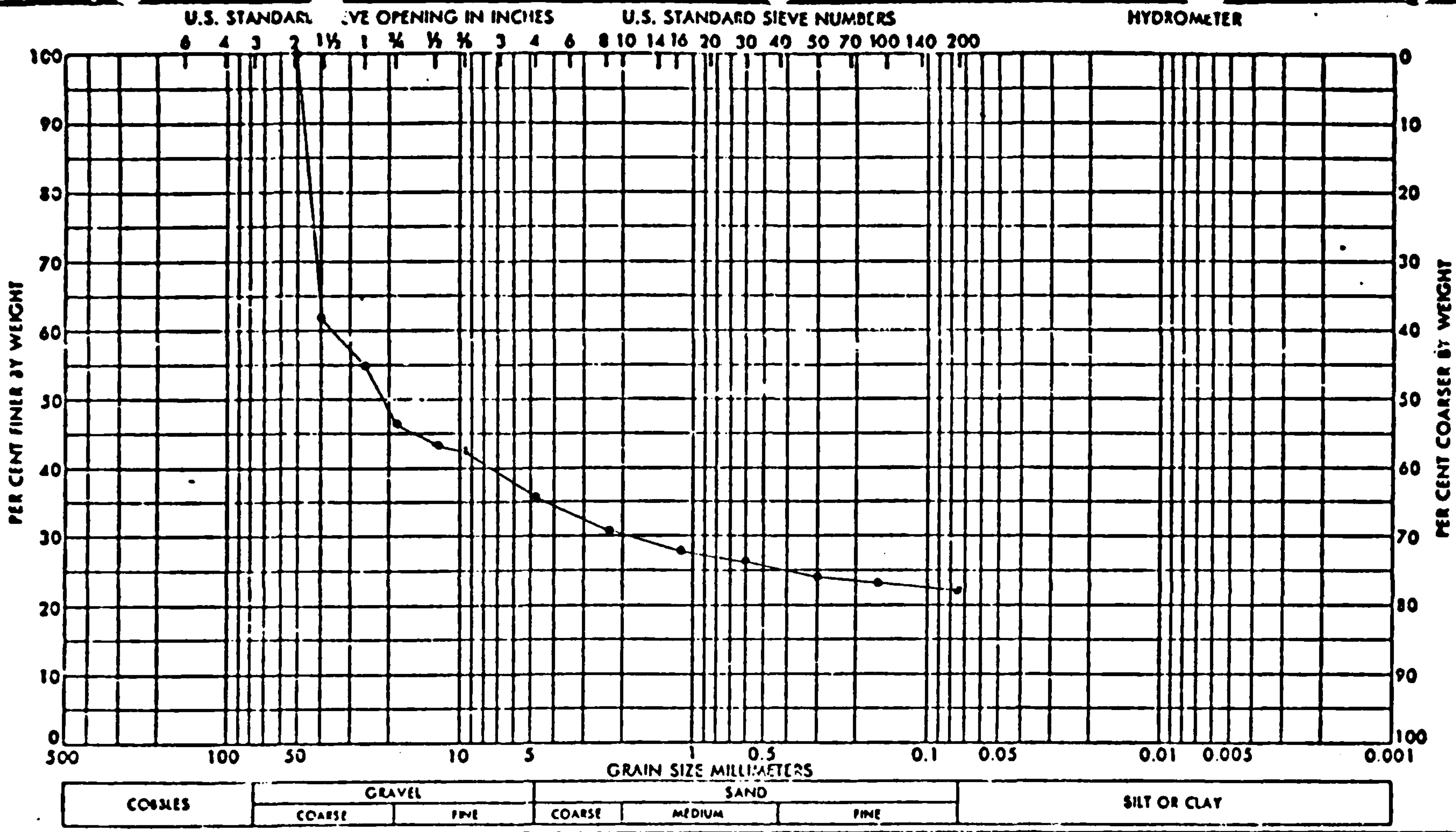
will require a prolonged period of time. While seepage control by means of the methods discussed in the text will certainly result in improved stability, it must be recognized that under the most ideal groundwater conditions, and even considering the highest probable shear strength parameters for the clay, the margin of safety for stability of the wall can not approach a magnitude which would be considered acceptable by modern design practices.

L. E. Wilson, Jr., P.E.

SUMMARY OF LABORATORY TEST RESULTS

Hole No.	Sample No.	Depth of Elevation	Natural Moisture (%)	Specific Gravity	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Unconfined Compressive Strength PSF	Triaxial Shear Test: Deviator Stress PSF	Triaxial Shear Test: Lateral Pressure PSI	Soil Description
1		637.5		2.67	36	17				Unified Soil Classification - CL Clay, silty, tan
4		628.0		2.54	77	52				Unified Soil Classification - CH Clay, light-brown
5		633.0		2.63	58	31				Unified Soil Classification - CH Clay, dark-brown with rock fragments
6		641.4		2.60	62	29				Unified Soil Classification - CH Clay, dark-brown
8		642-643		2.68	49	22				Unified Soil Classification - CL Clay, slightly silty, dark-brown
9	A	632.0	12.1		37	13				Unified Soil Classification - CL Clay, silty, brown
9	B	631.0	15.8		36	16				Unified Soil Classification - CL Clay, silty, tan
9	C	630.5	13.4		32	14				Unified Soil Classification - CL Clay, silty, brown
10		641.8	9.4		35	15				Unified Soil Classification - CL Clay, silty, tan

8th Avenue Reservoir
Metro Water & Sewerage Services
Project No. 75-090



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

GRADATION CURVES

PROJECT 8th Avenue Reservoir

Metro Water & Sewerage Services

AREA

BORING NO. 1, Elevation 637.5

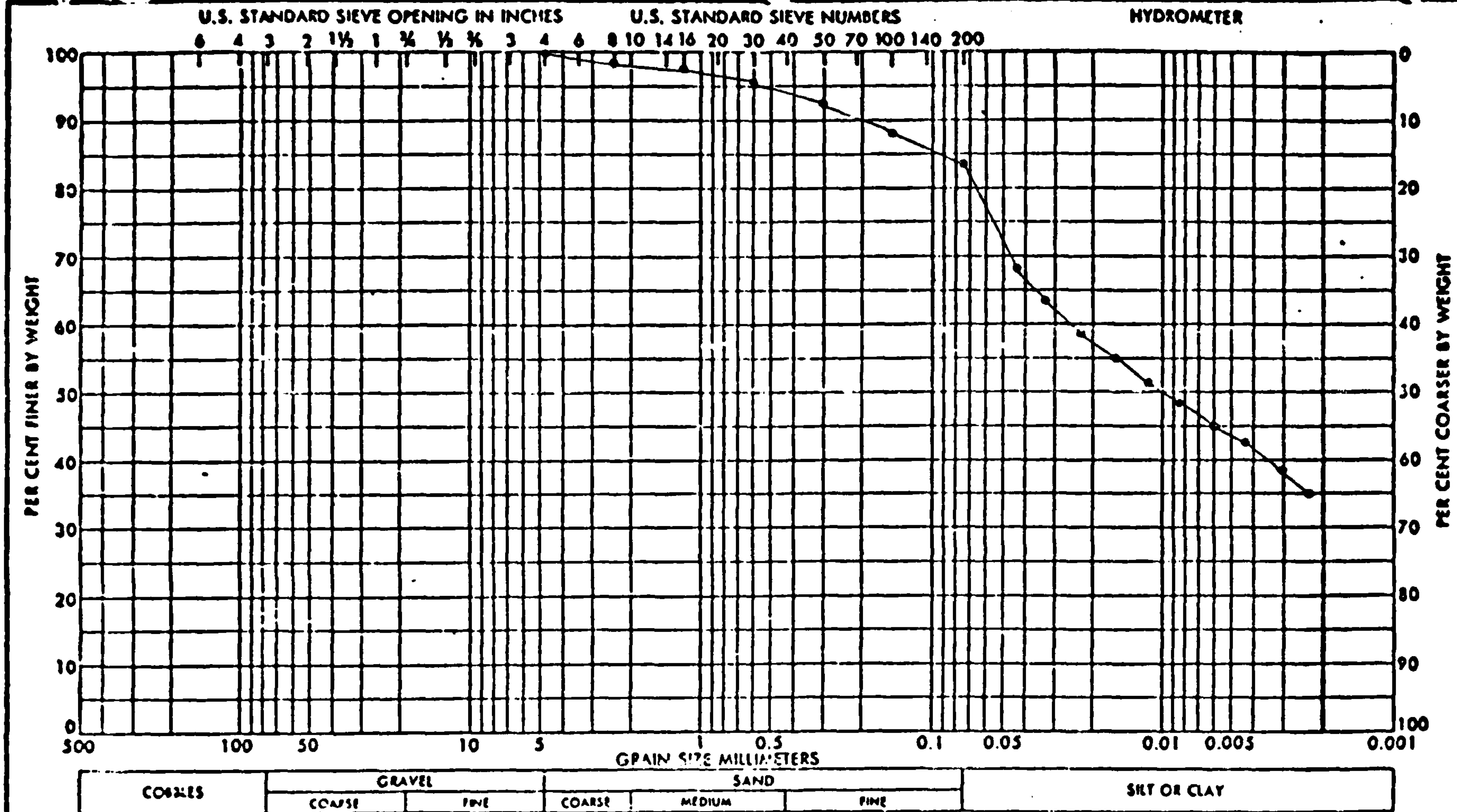
DATE July 15, 1975

GEOLOGIC ASSOCIATES, INC.

LABORATORY DIVISION

Franklin, Tennessee

Project No. 75-090



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

GRADATION CURVES

PROJECT 8th Avenue Reservoir

Metro Water & Sewerage Services

AREA

BORING NO. 4, Elevation 628

DATE July 14, 1975

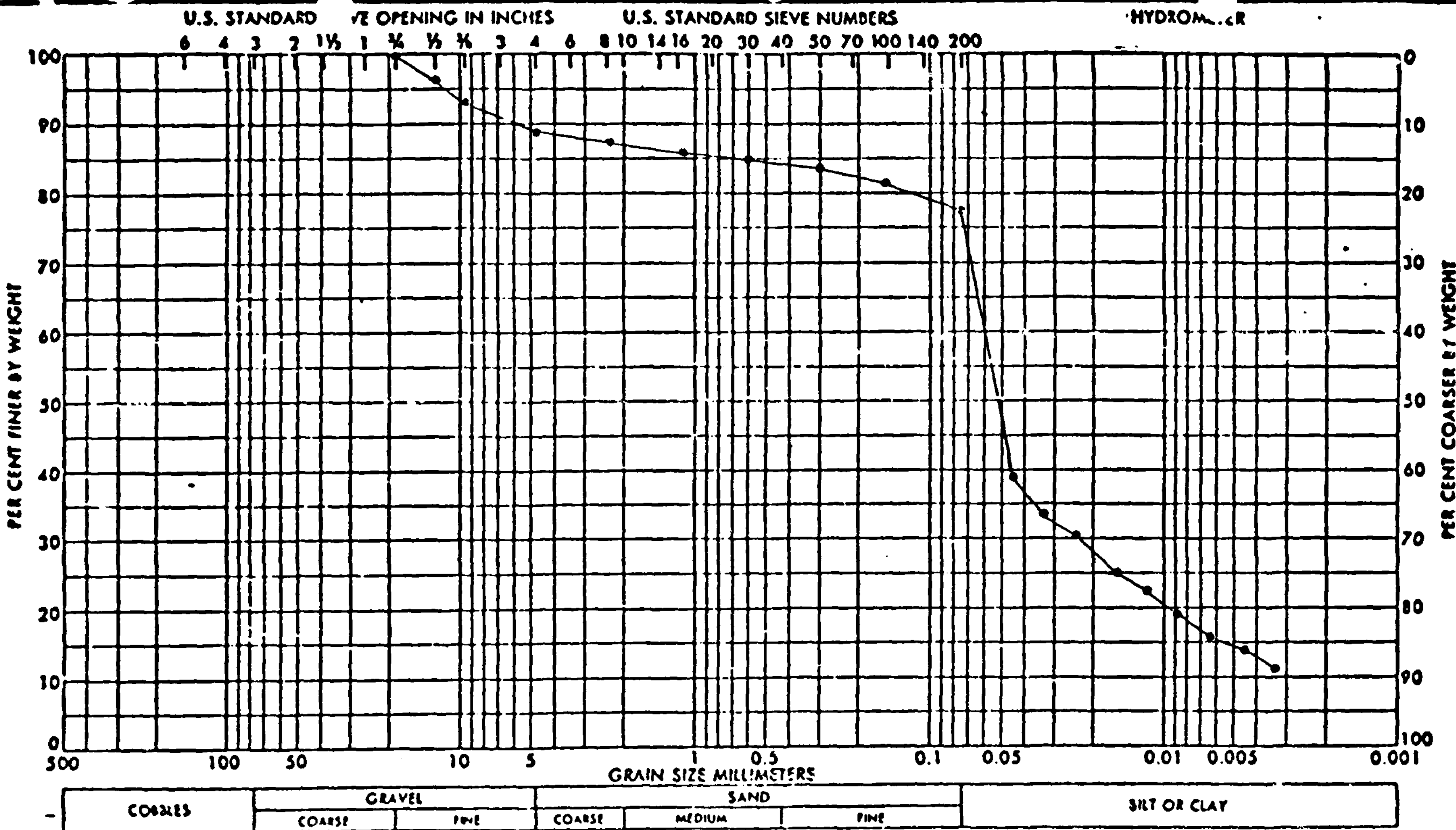
Project No. 75-090

GEOLOGIC ASSOCIATES, INC.

LABORATORY DIVISION

Franklin, Tennessee [illegible]

[illegible], Tennessee [illegible]



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

GRADATION CURVES

PROJECT 8th Avenue Reservoir

Metro Water & Sewerage Services

AREA

BORING NO. 5, Elevation 633

DATE July 15, 1975

Project No. 75-090

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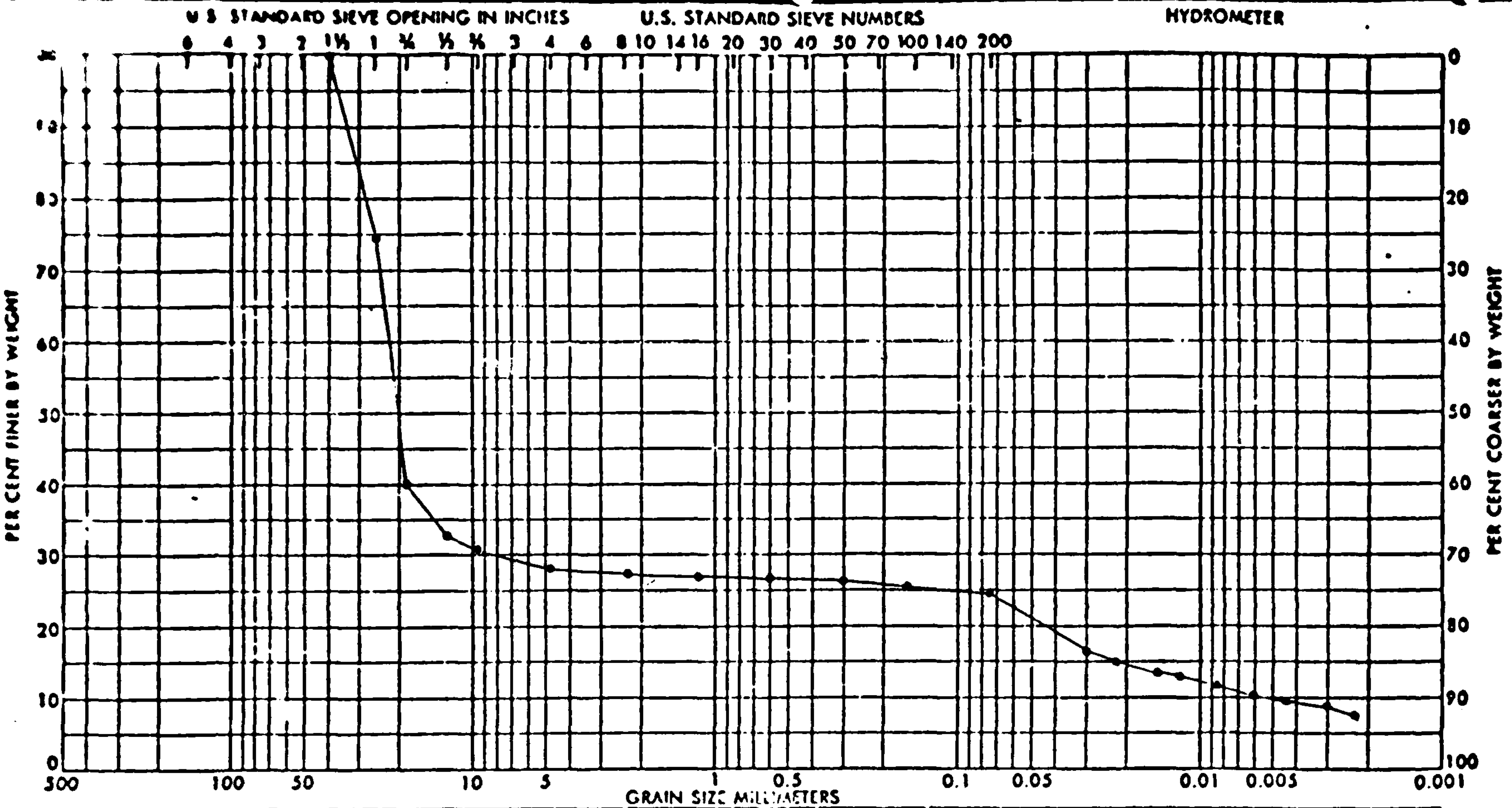
LABORATORY DIVISION

Franklin, Tennessee 615 794-3551

Knoxville, Tennessee 615 [illegible]



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

GRADATION CURVES

PROJECT 8th Avenue Reservoir

Metro Water & Sewerage Services

AREA

BORING NO. 6, Elevation 641.4

DATE July 14, 1975

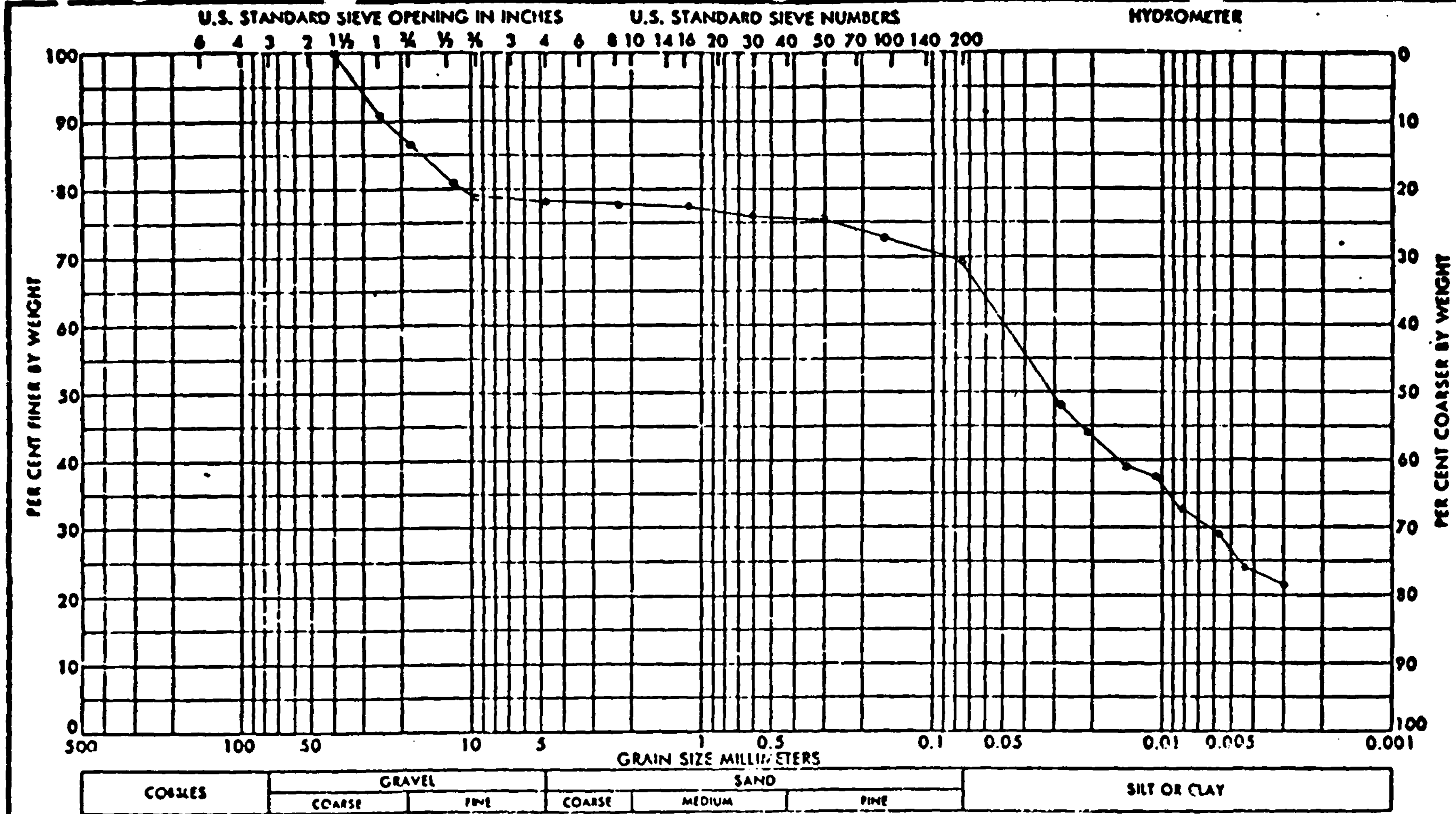
GEOLOGIC ASSOCIATES, INC.

LABORATORY DIVISION

Franklin, Tennessee 615 794-3898

Knoxville, Tennessee 615 [illegible]

Project No. 75-090



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

GRADATION CURVES

PROJECT 8th Avenue Reservoir

Metro Water & Sewerage Services

AREA

BORING NO. 8, Elevation 642 - 643

DATE July 15, 1975

Project No. 75-090

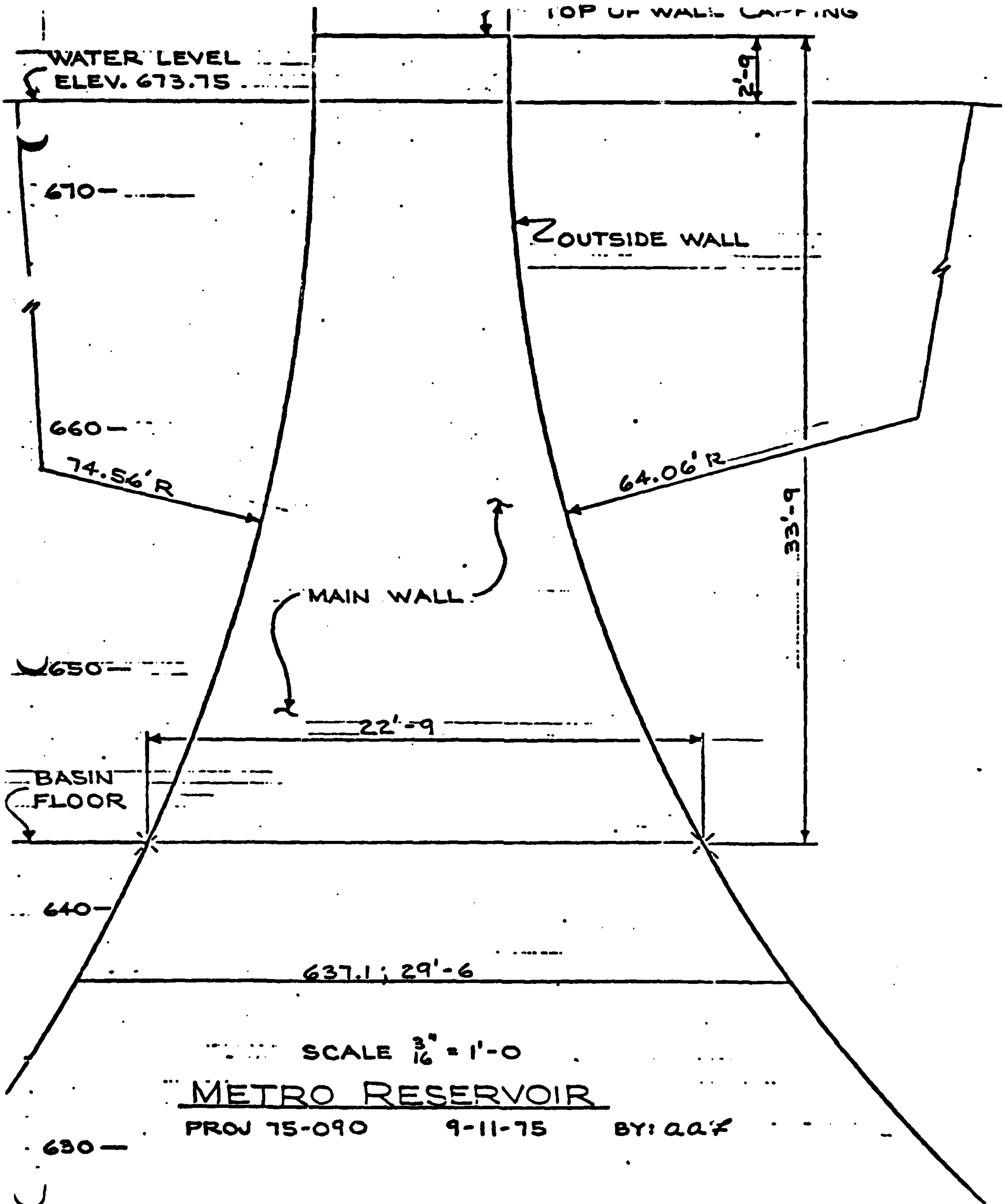
GEOLOGIC ASSOCIATES, INC.

LABORATORY DIVISION

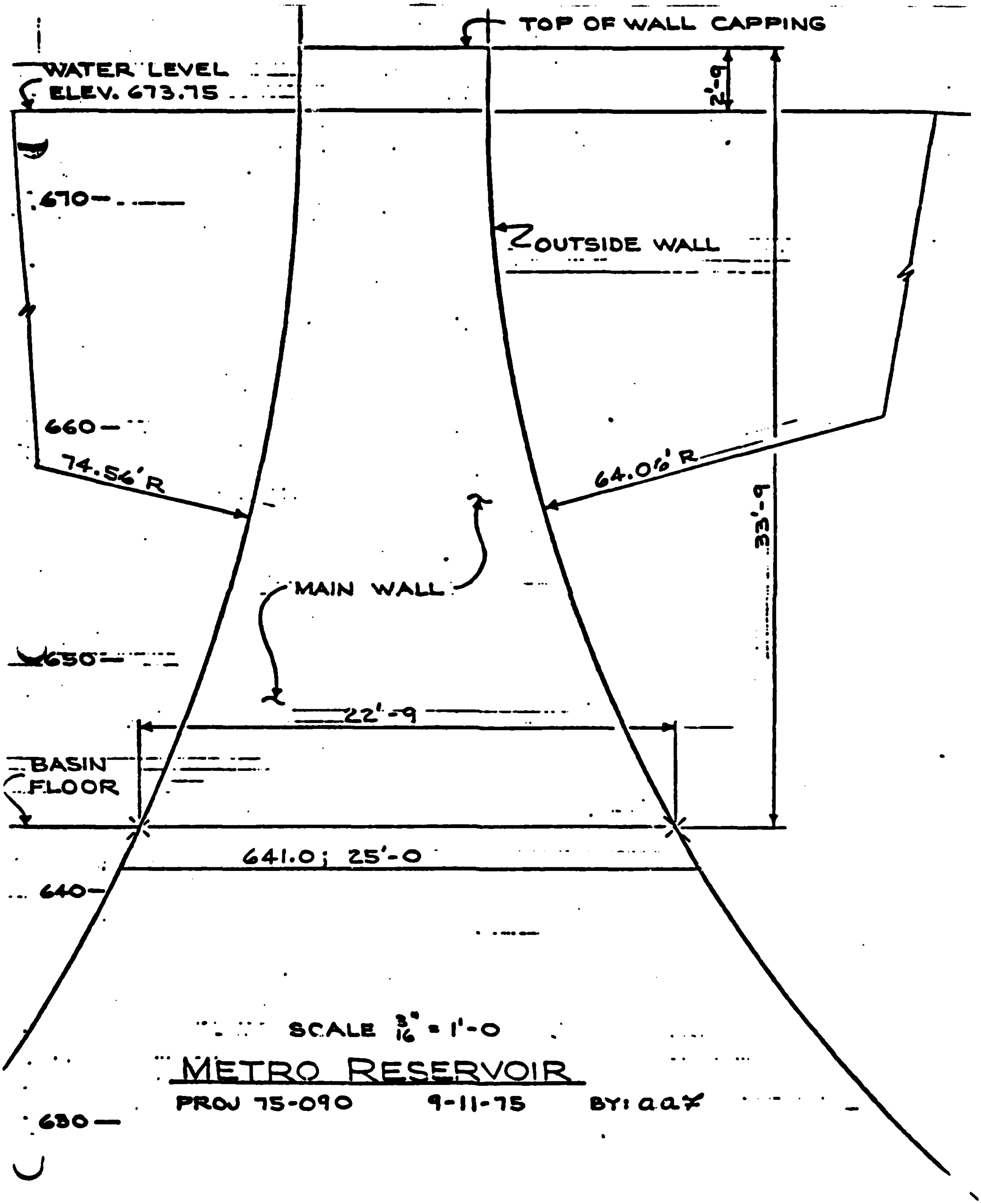
Franklin, Tennessee 615 794-3598

APPENDIX B

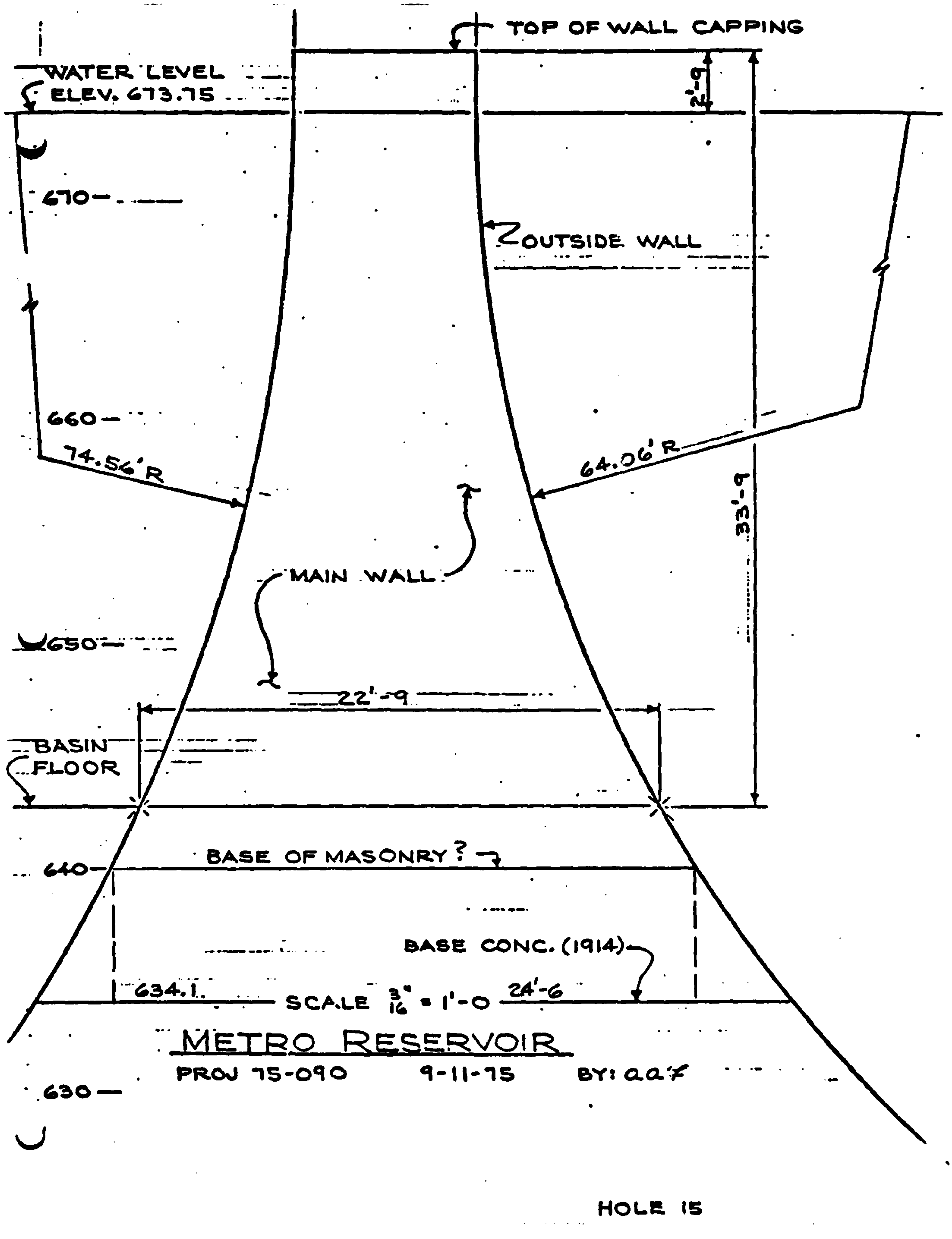
TYPICAL WALL SECTIONS

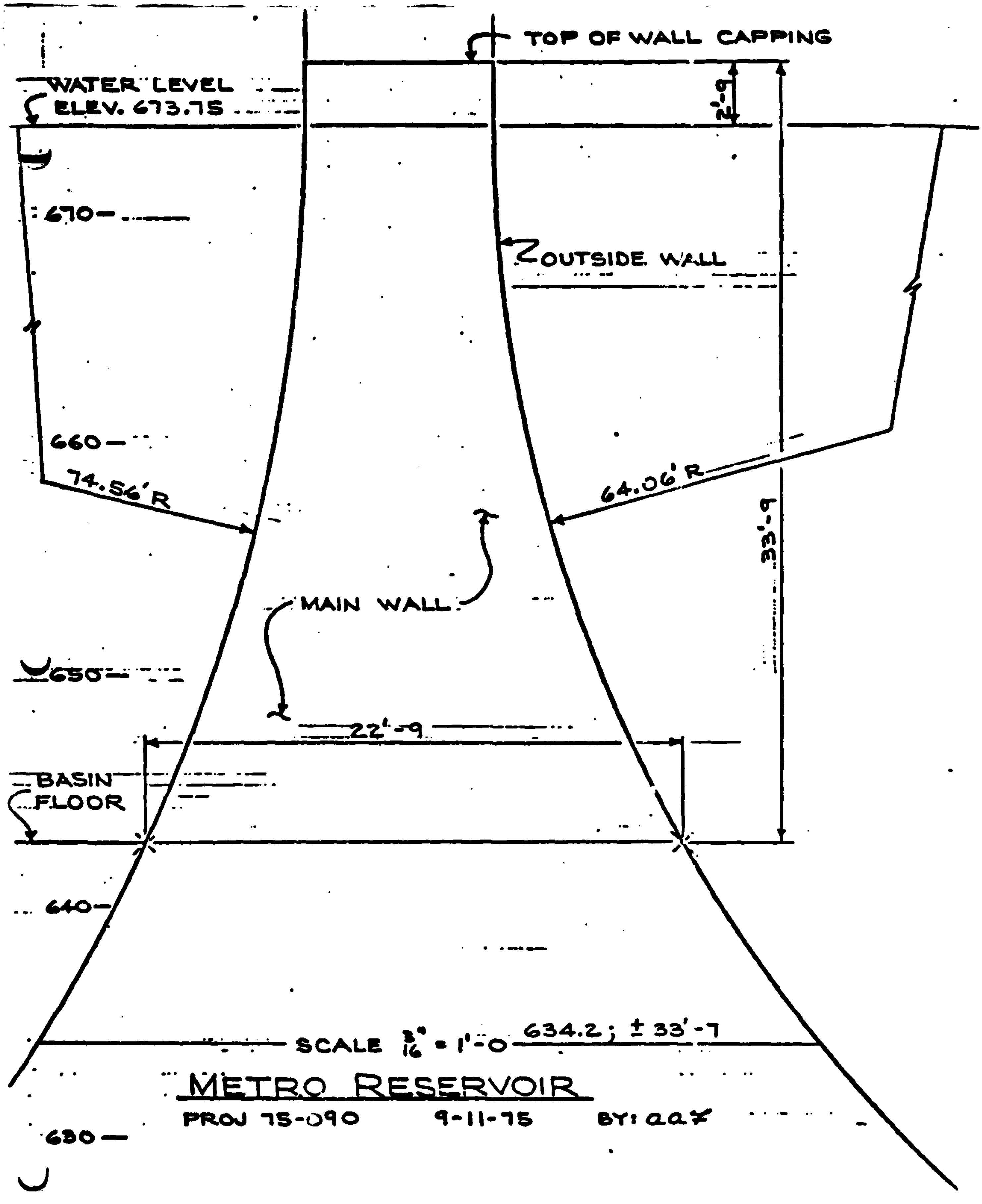


ESTIMATED ELEV. BASE OF MASONRY AND WIDTH OF FOUNDATION AT HOLE 13

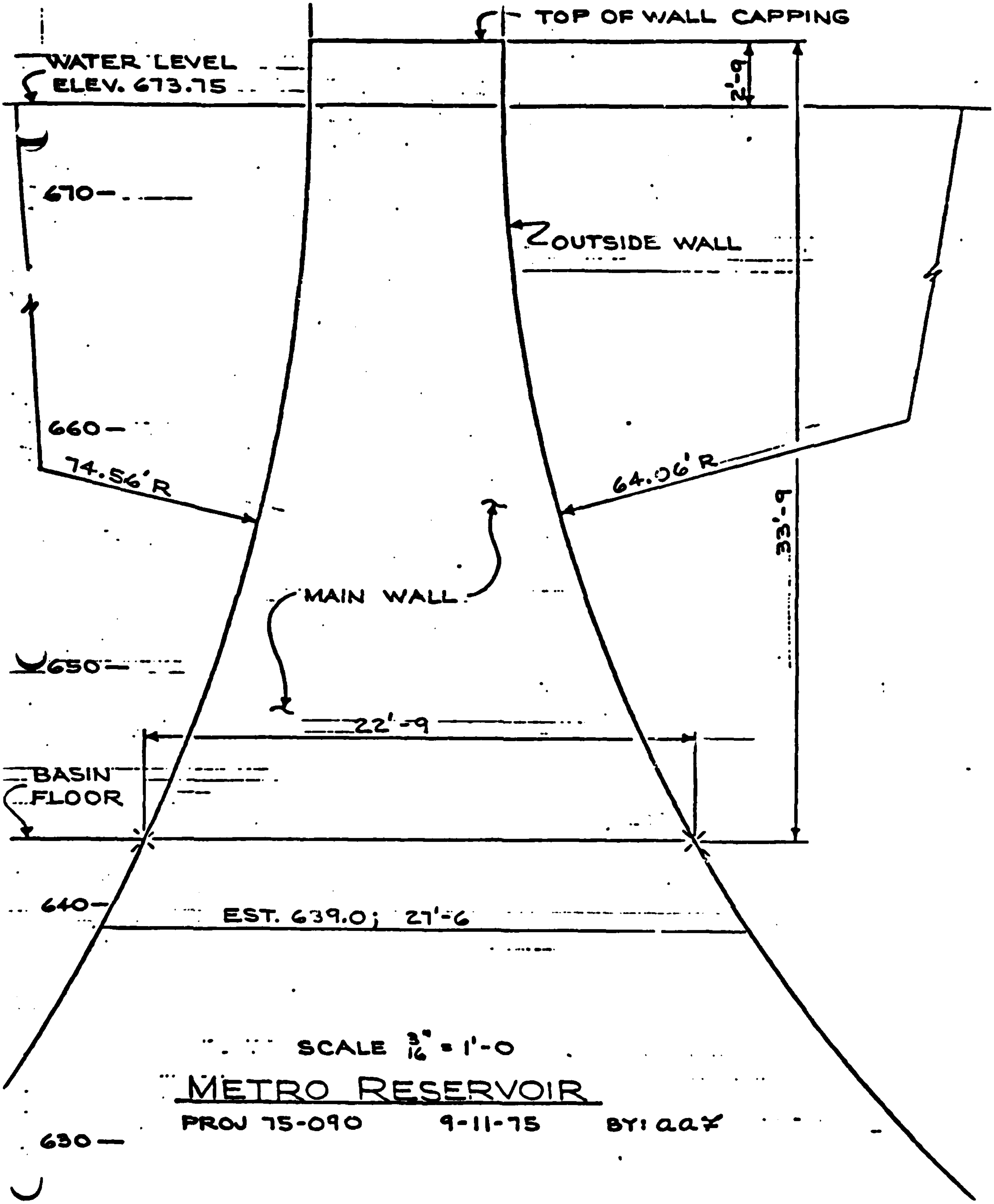

TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670
OUTSIDE WALL
660
74.56'R
64.06'R
33'-9
MAIN WALL
650
22'-9
BASIN
FLOOR
641.0; 25'-0
640
SCALE 3/16" = 1'-0
METRO RESERVOIR
PROJ 75-090
9-11-75
BY: aa
630

HOLE 14


TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670
OUTSIDE WALL
660
74.56' R
64.06' R
33'-9
MAIN WALL
650
22'-9
BASIN
FLOOR
BASE OF MASONRY ?
640
BASE CONC. (1914)
634.1
SCALE 3/16" = 1'-0
24'-6
METRO RESERVOIR
PROJ 75-090
9-11-75
BY: aaf
630
HOLE 15


TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670
OUTSIDE WALL
660
74.56' R
64.06' R
33'-9
MAIN WALL
650
22'-9
BASIN
FLOOR
640
SCALE 3/16" = 1'-0
634.2; ± 33'-7
METRO RESERVOIR
PROJ 75-090
9-11-75
BY: aa
630

HOLE 16

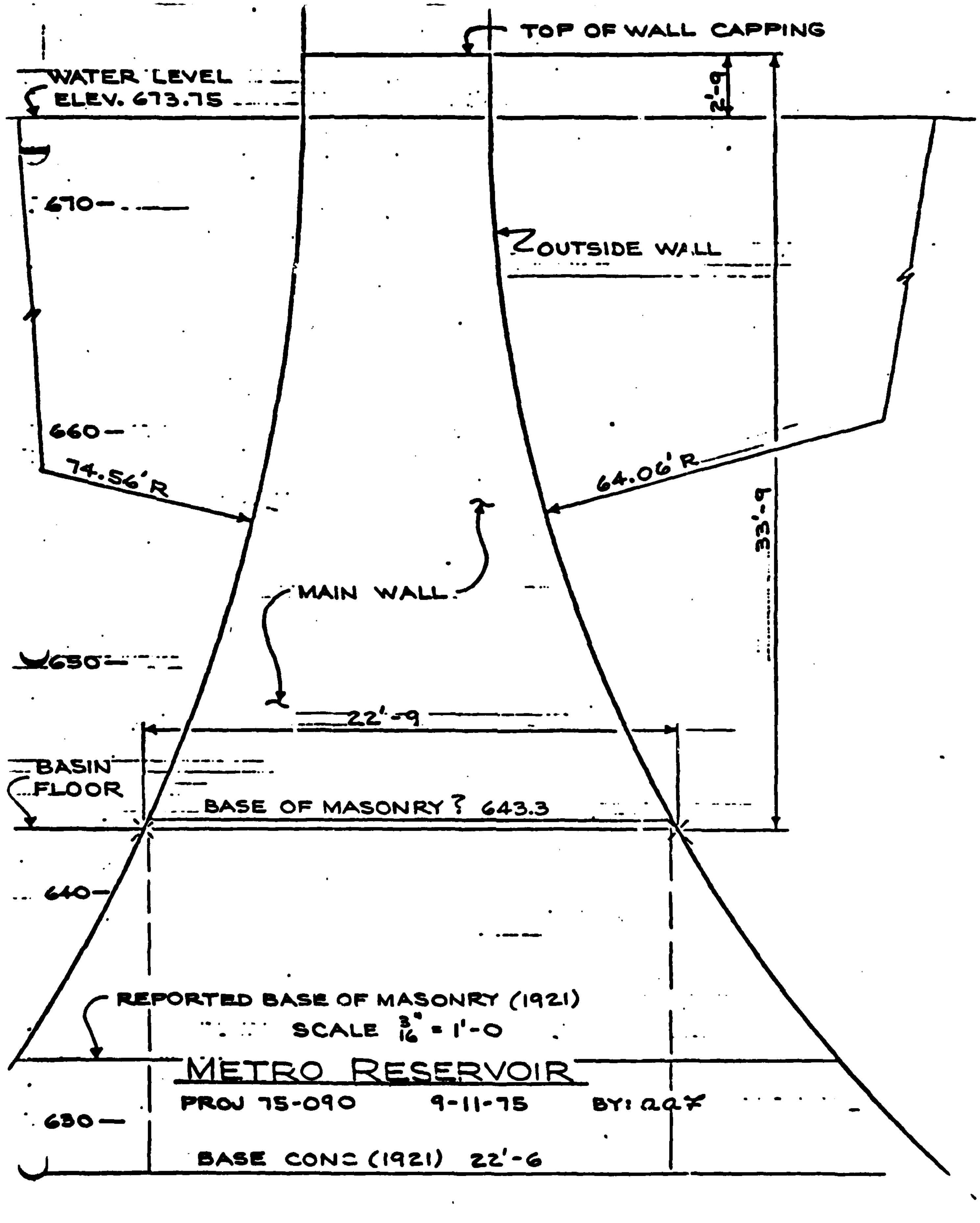


SCALE $\frac{3}{16}$" = 1'-0

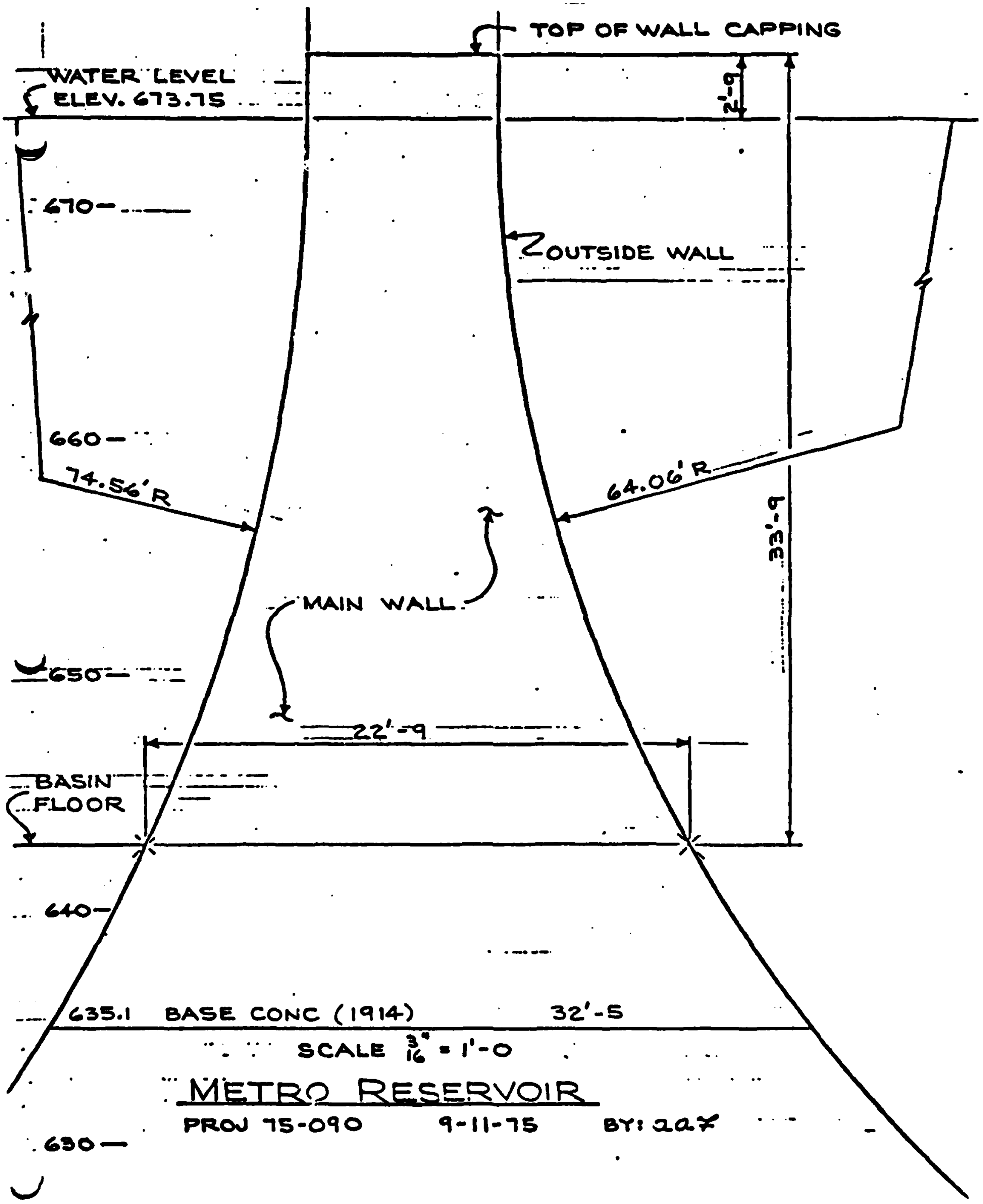
METRO RESERVOIR

PROJ 75-090 9-11-75 BY: aa7

HOLE 17


TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670
OUTSIDE WALL
660
74.56' R
64.06' R
33'-9
MAIN WALL
650
22'-9
BASIN
FLOOR
BASE OF MASONRY ? 643.3
640
REPORTED BASE OF MASONRY (1921)
SCALE 3/16" = 1'-0
METRO RESERVOIR
PROJ 75-090
9-11-75
BY:
630
BASE CONC (1921) 22'-6

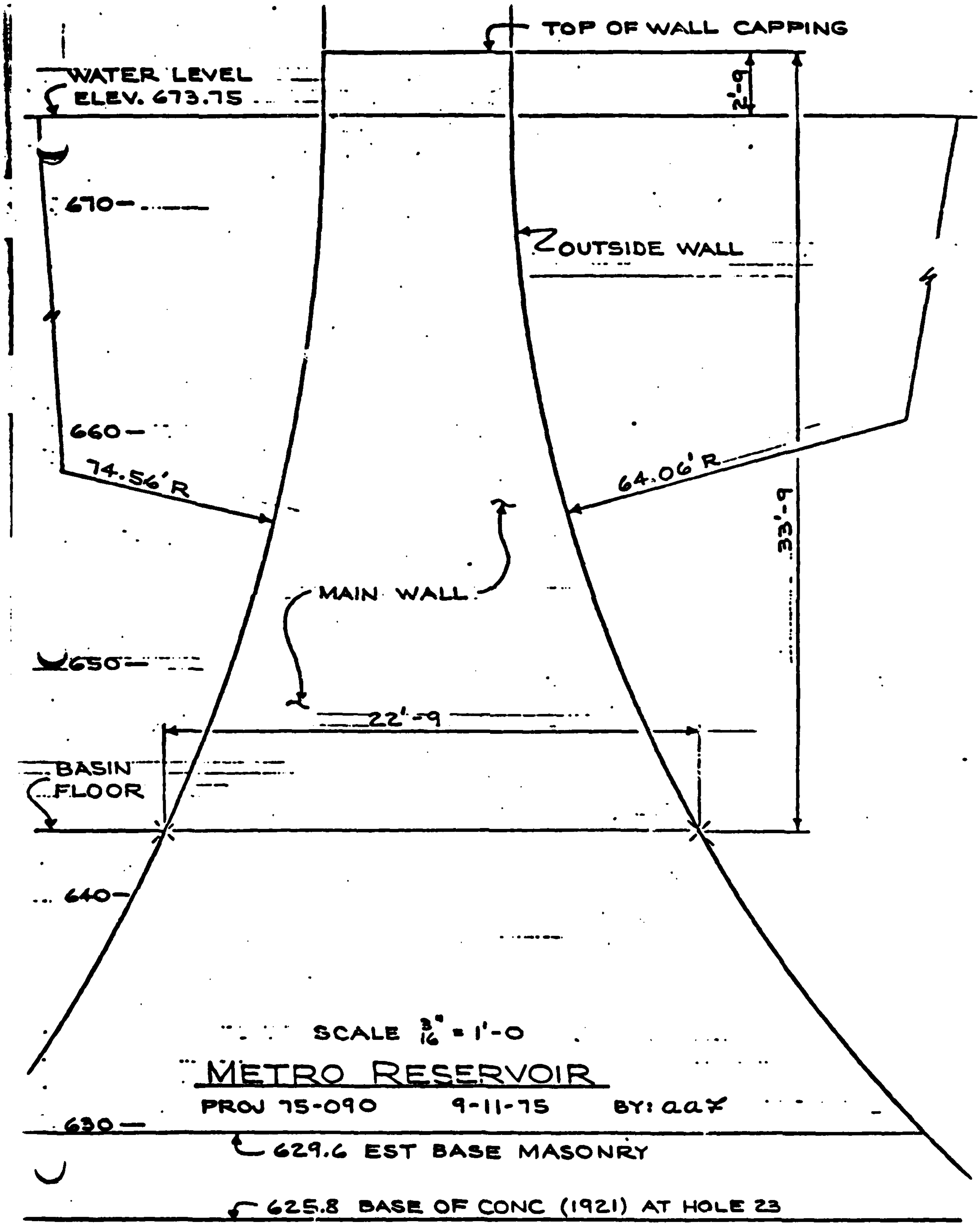
HOLE 21

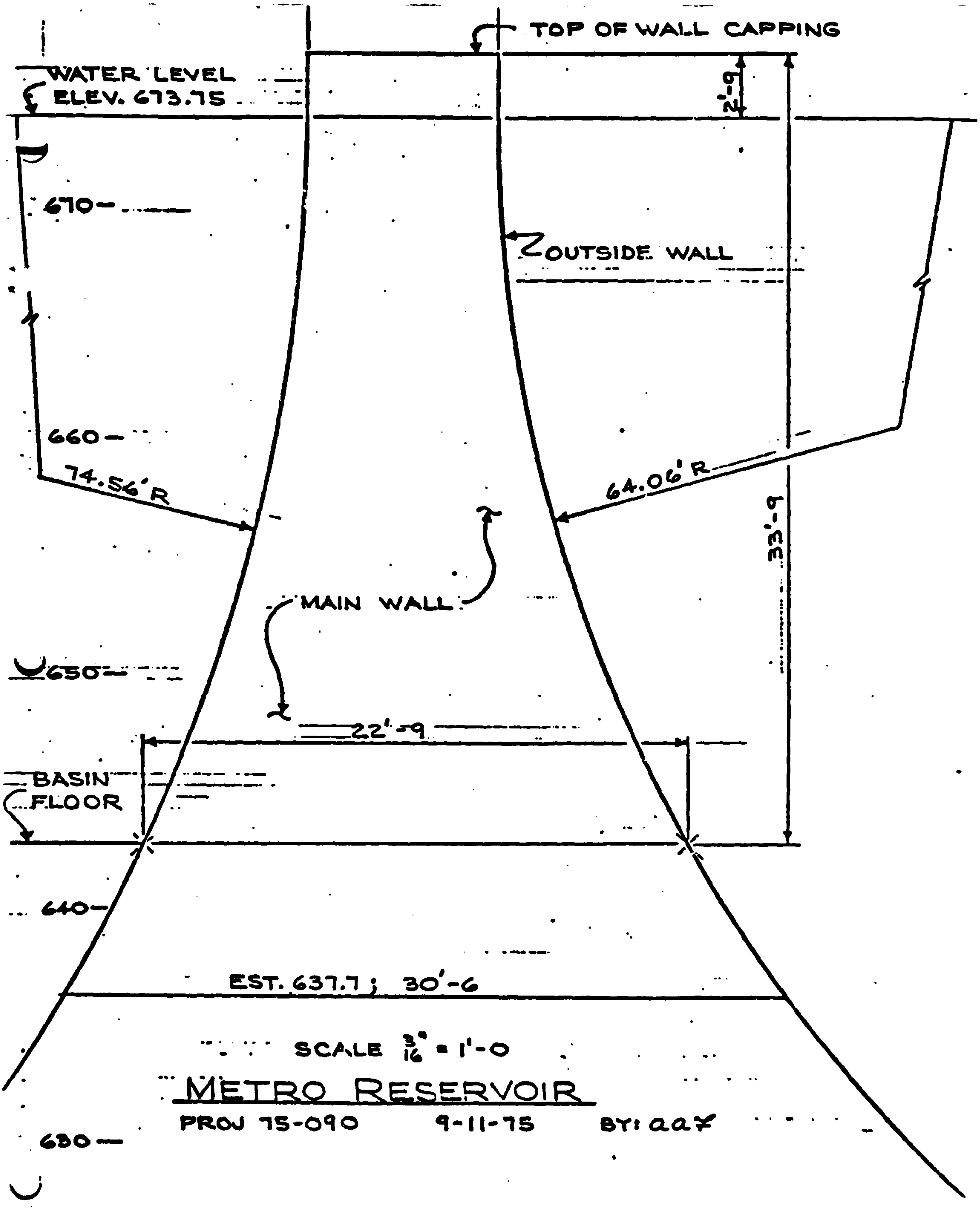


METRO RESERVOIR

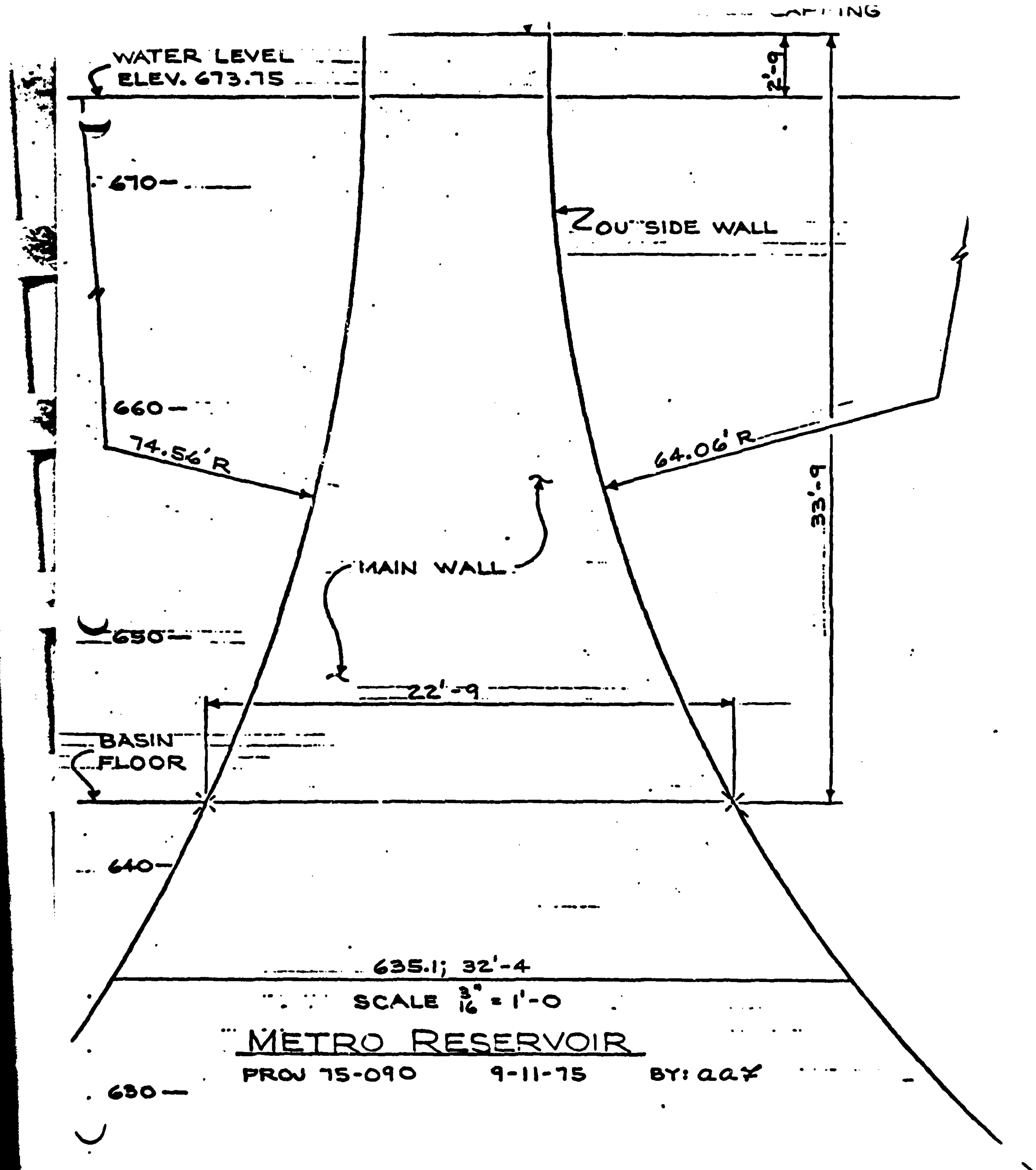
PROJ 75-090 9-11-75 BY: ααF

HOLE 22


TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670—
OUTSIDE WALL
660—
74.56'R
64.06'R
33'-9
MAIN WALL
650—
22'-9
BASIN
FLOOR
640—
SCALE 3/16" = 1'-0
METRO RESERVOIR
PROJ 75-090
9-11-75
BY: aa7
630—
629.6 EST BASE MASONRY
625.8 BASE OF CONC (1921) AT HOLE 23


TOP OF WALL CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670—
OUTSIDE WALL
660—
74.56'R
64.06'R
33'-9
MAIN WALL
650—
22'-9
BASIN
FLOOR
640—
EST. 637.7 ; 30'-6
SCALE 3/16" = 1'-0
METRO RESERVOIR
PROJ 75-090 9-11-75 BY: aa7
630—

HOLE 24


CAPPING
WATER LEVEL
ELEV. 673.75
2'-9
670—
OUTSIDE WALL
660—
74.56'R
64.06'R
33'-9
MAIN WALL
650—
22'-9
BASIN
FLOOR
640—
635.1; 32'-4
SCALE 3/16" = 1'-0
METRO RESERVOIR
PROJ 75-090 9-11-75 BY: aa?
630—

HOLE 26

APPENDIX C

INCLINOMETER RECORDS

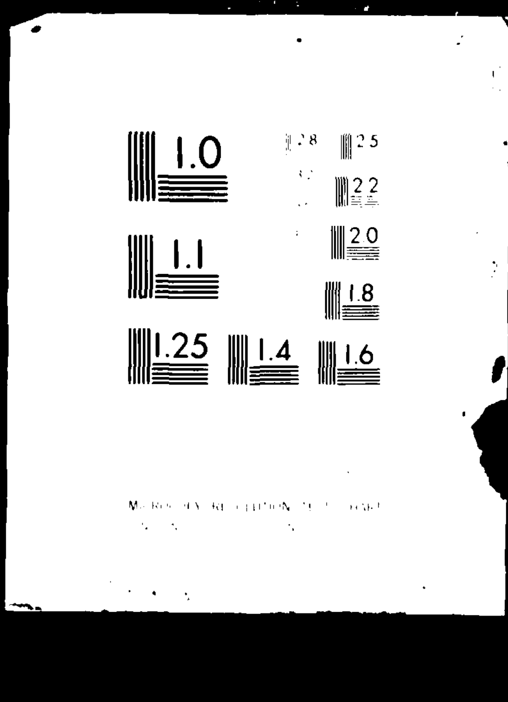
AD-A108 470 TENNESSEE STATE DEPT OF CONSERVATION NASHVILLE DIV O--ETC F/G 13/13
NATIONAL PROGRAM OF INSPECTION OF NON-FEDERAL DAMS, TENNESSEE. --ETC(U)
SEP 81 P F BLUHM DACW62-81-C-0056

UNCLASSIFIED NL

3 OF 4
AD A
108470

3 OF 4

AD A
108470



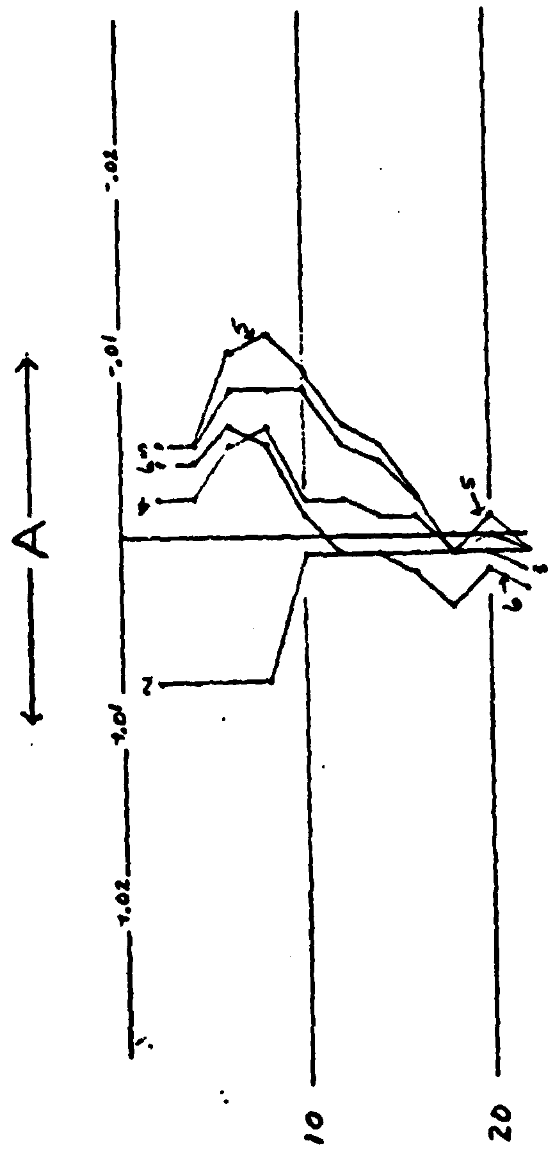
Inclinometer Records

FOR Hole 10 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975

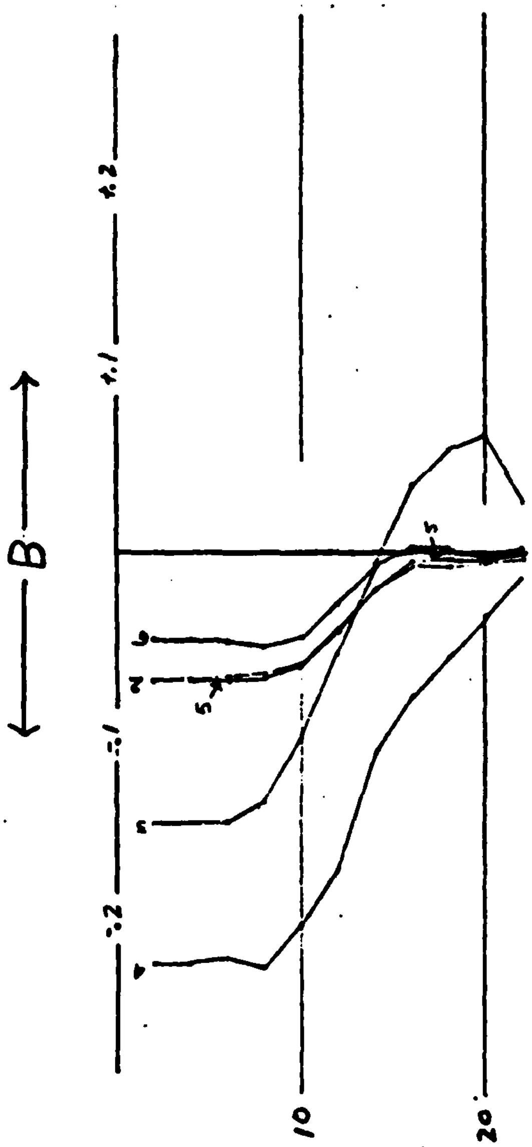


Inclinometer Records

FOR Hole 10 BY

Engineers & Geologists
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

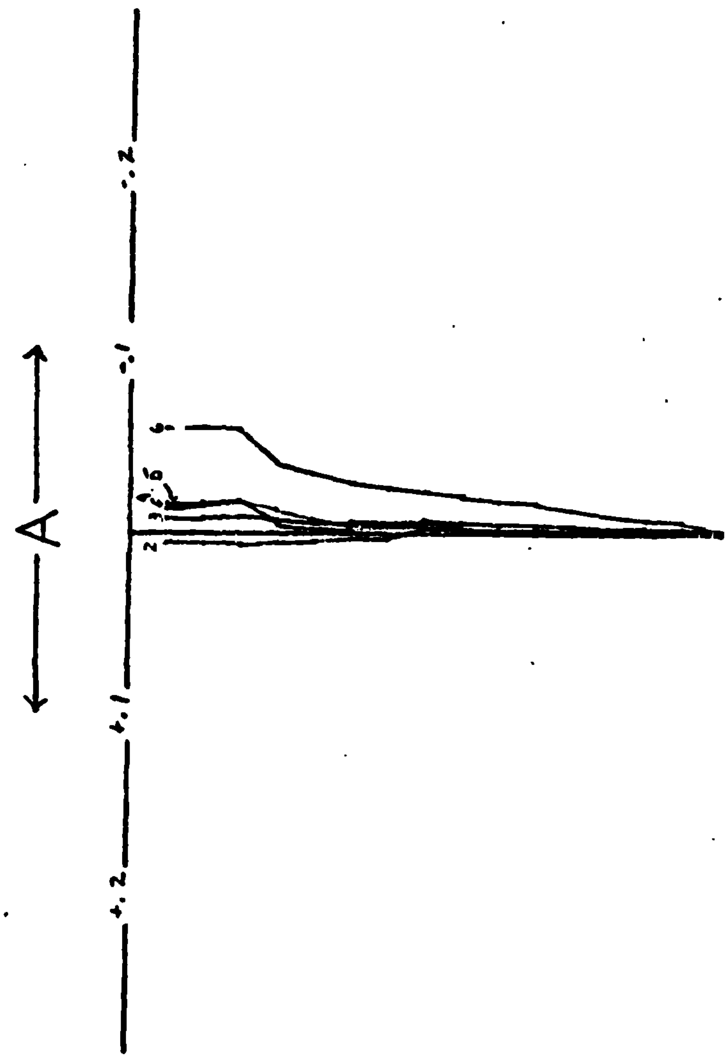
PROJECT NO. 75-090
DATE August 23, 1975



Inclinometer Records ENGINEERS & GEOLOGISTS PROJECT NO. 75-090

FOR Hole 11 BY FRANKLIN, TENNESSEE DATE August 23, 1975

KNOXVILLE, TENNESSEE

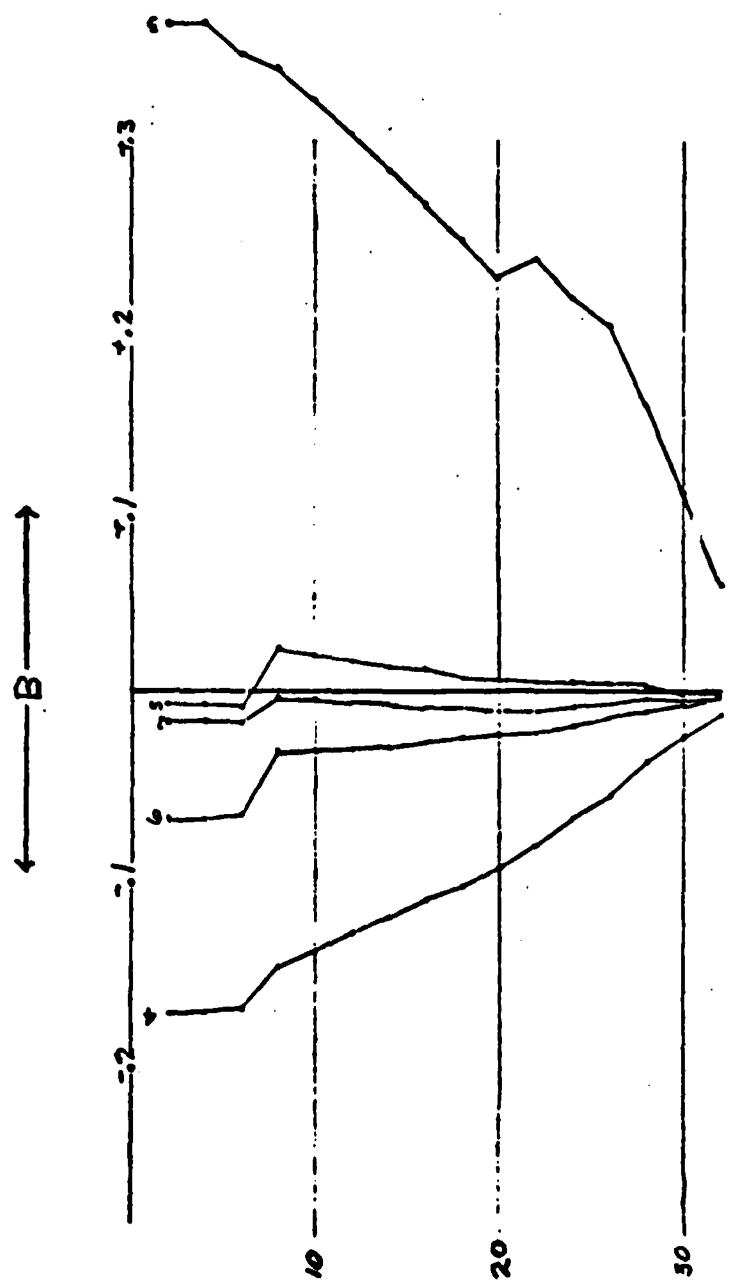


[illegible]nometer Records

FOR Hole 11 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

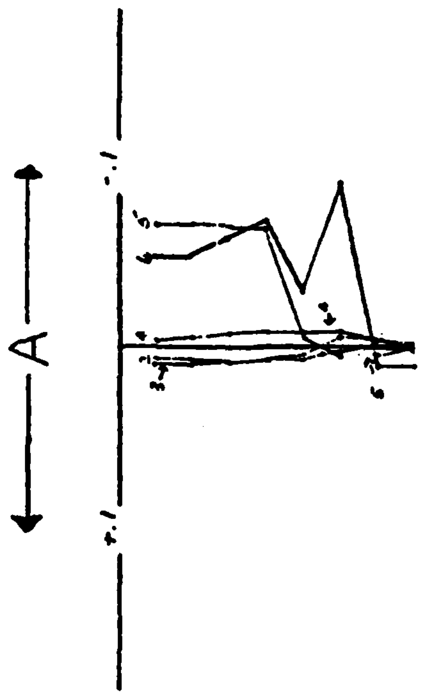
PROJECT NO. 75-090
DATE August 23, 1975



FOR Hole 12 BY

FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

DATE August 23, 1975



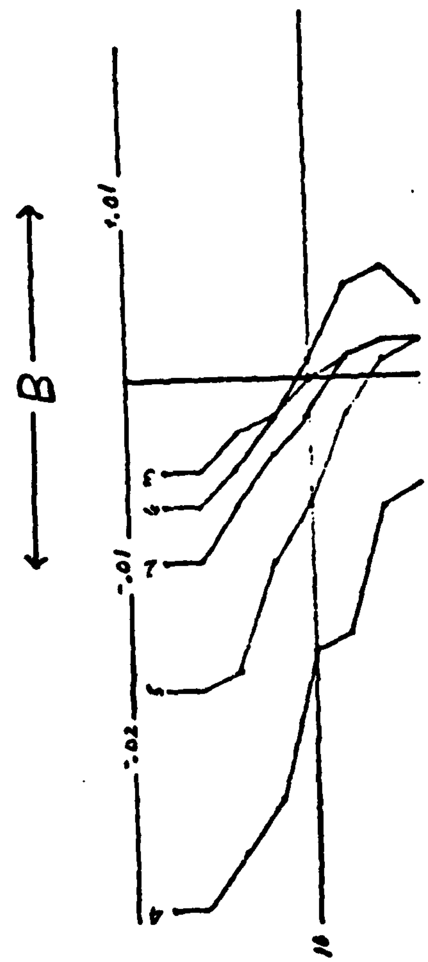
Inclinometer Records

FOR Hole 12 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975



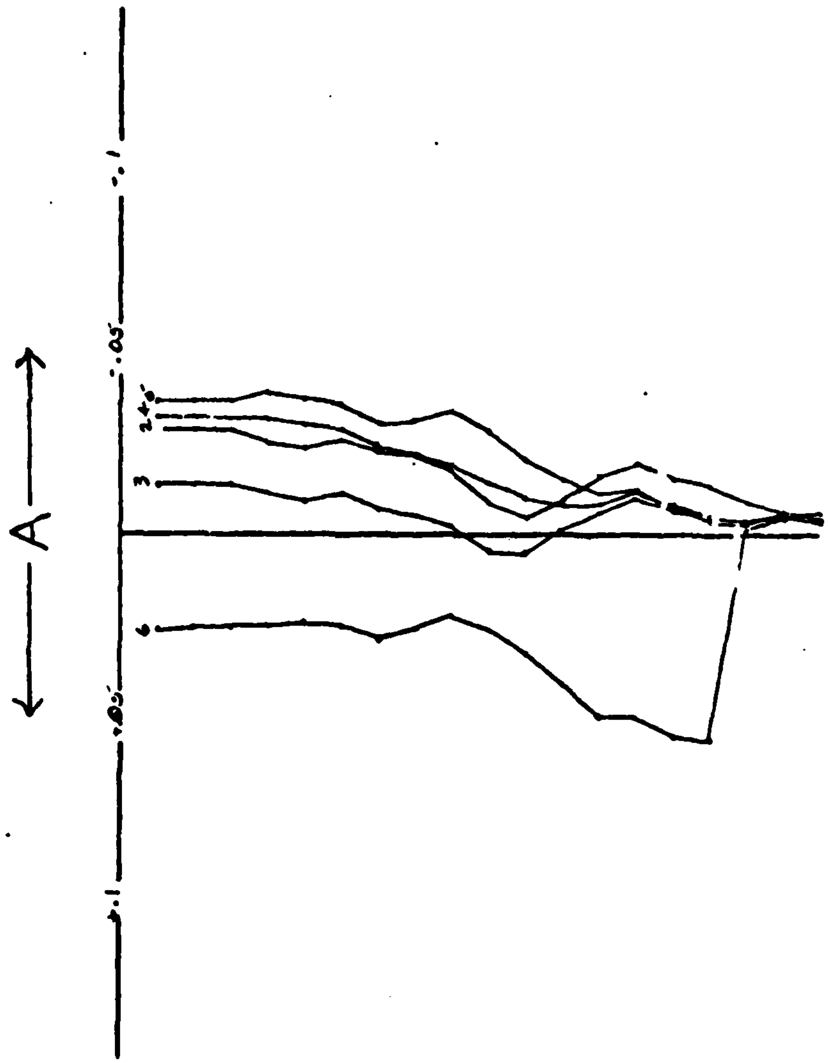
Inclinometer Records

FOR Hole 13 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975

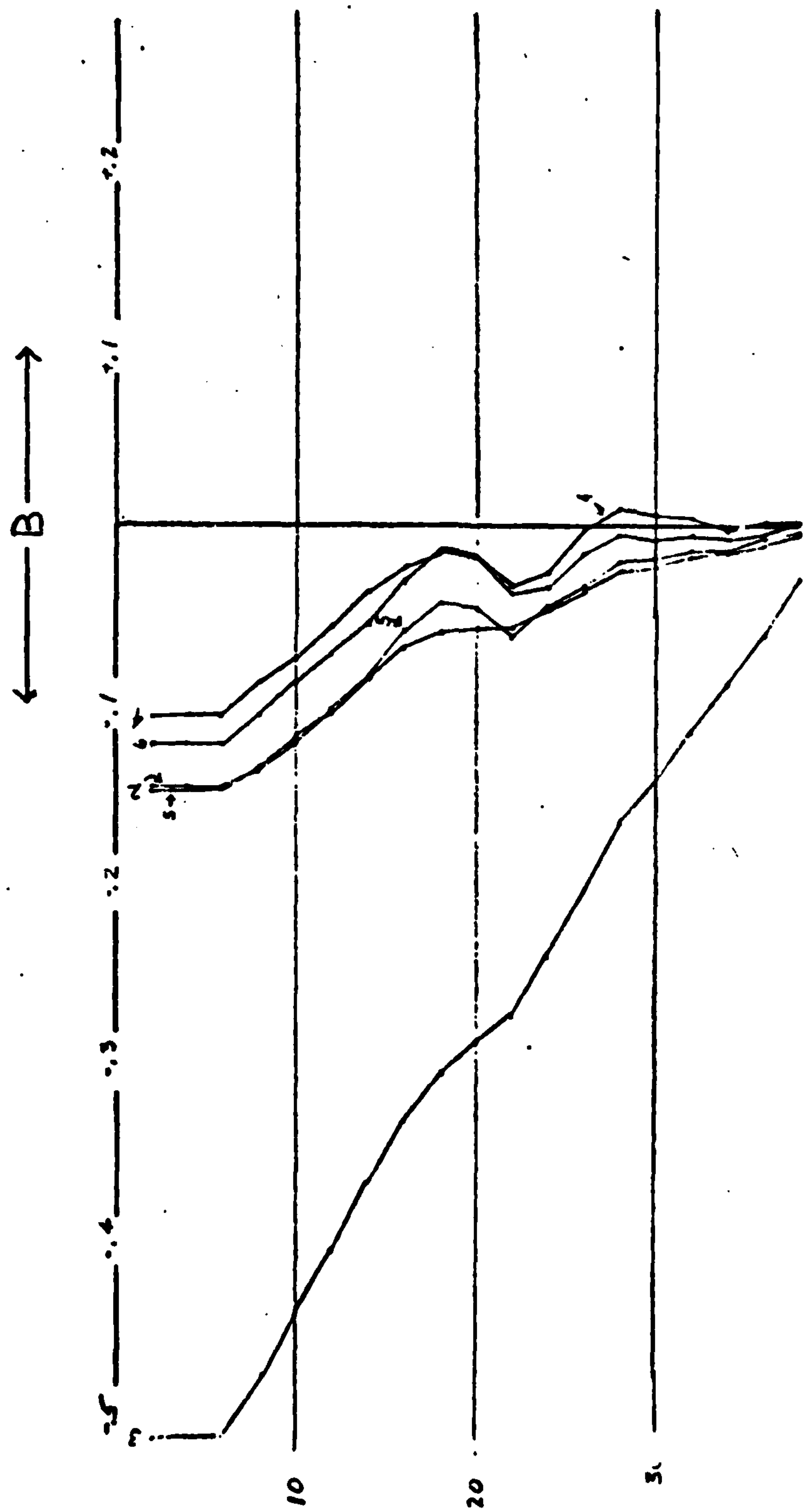


FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

FOR Hole 13 BY

PROJECT NO. [illegible]

DATE August 23, 1975



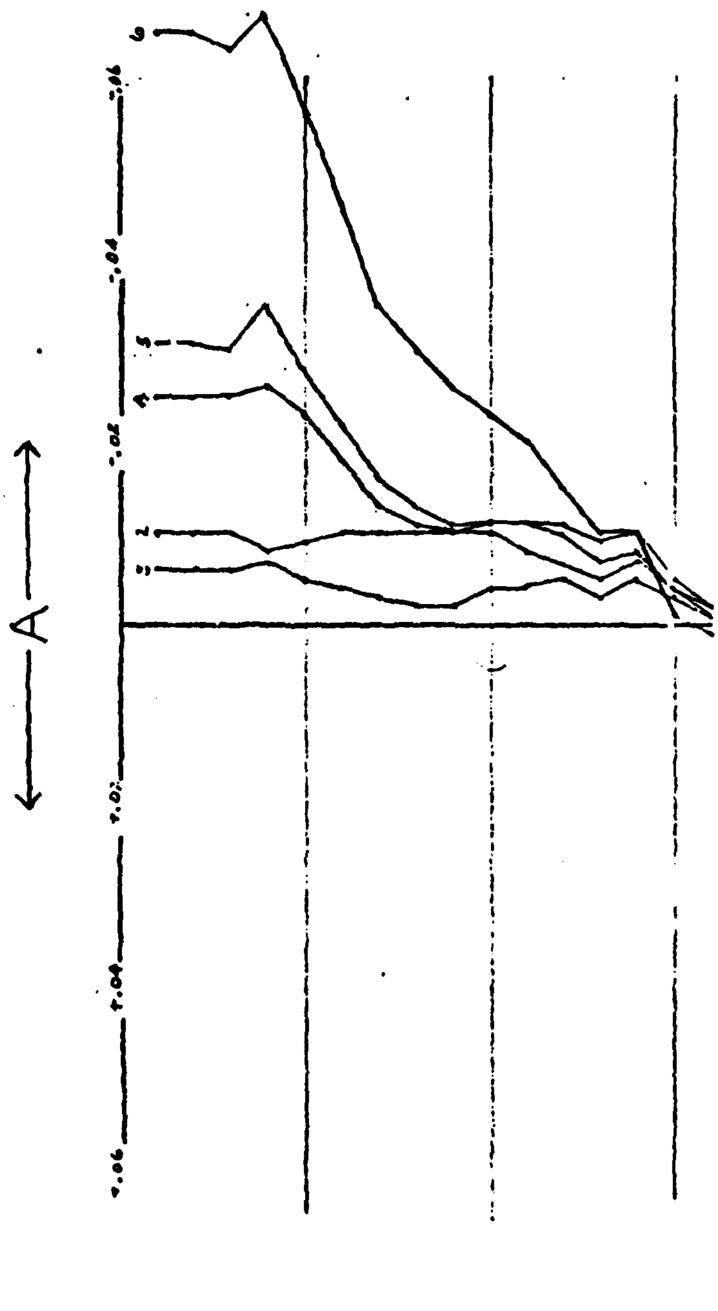
Inclinometer Records

FOR Hole 14 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975



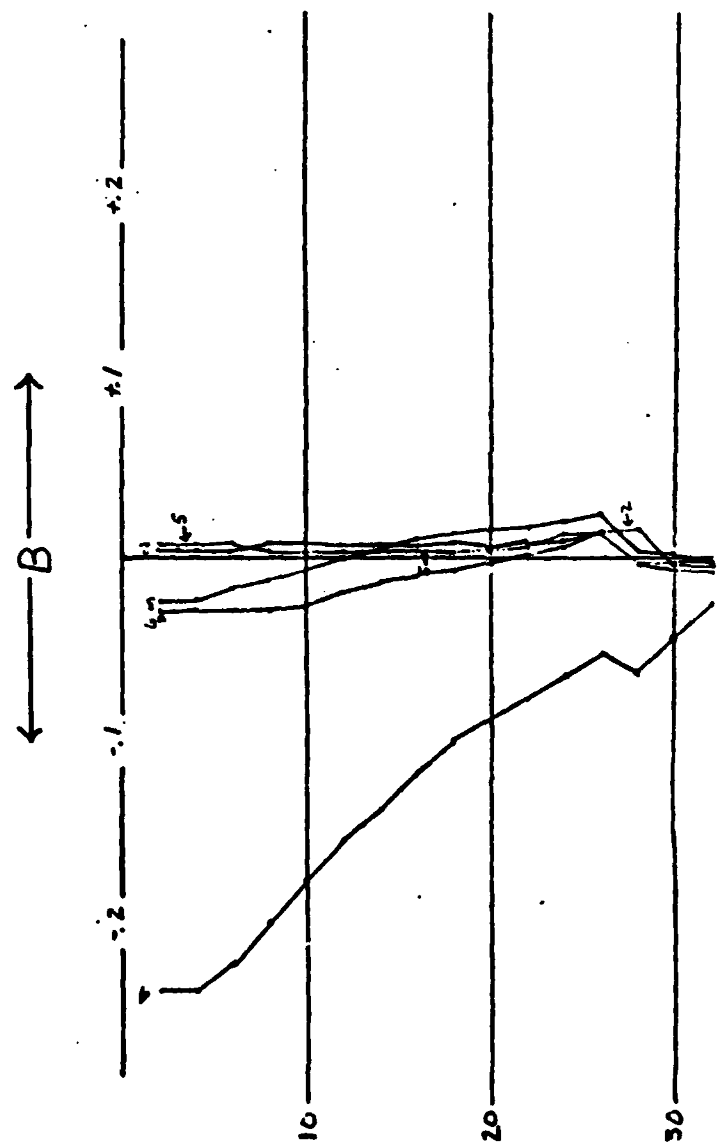
Inclinometer Records

FOR Hole 14 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975



PROJECT Inclinometer Records

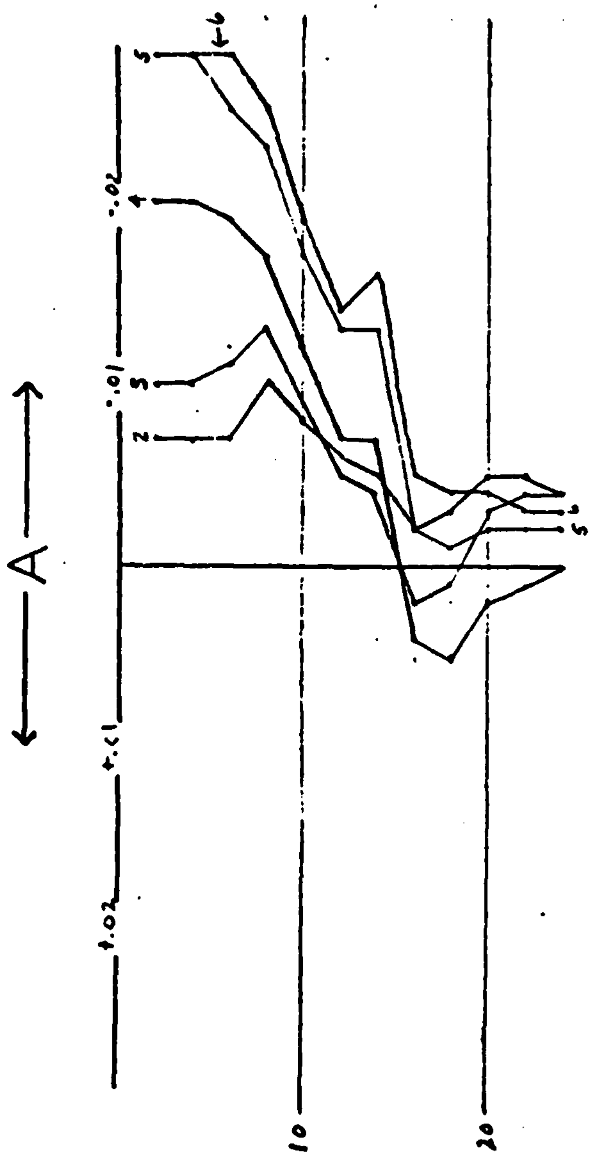
FOR Hole 15 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO. 11 OF

PROJECT NO. 75-090

DATE August 23, 1975



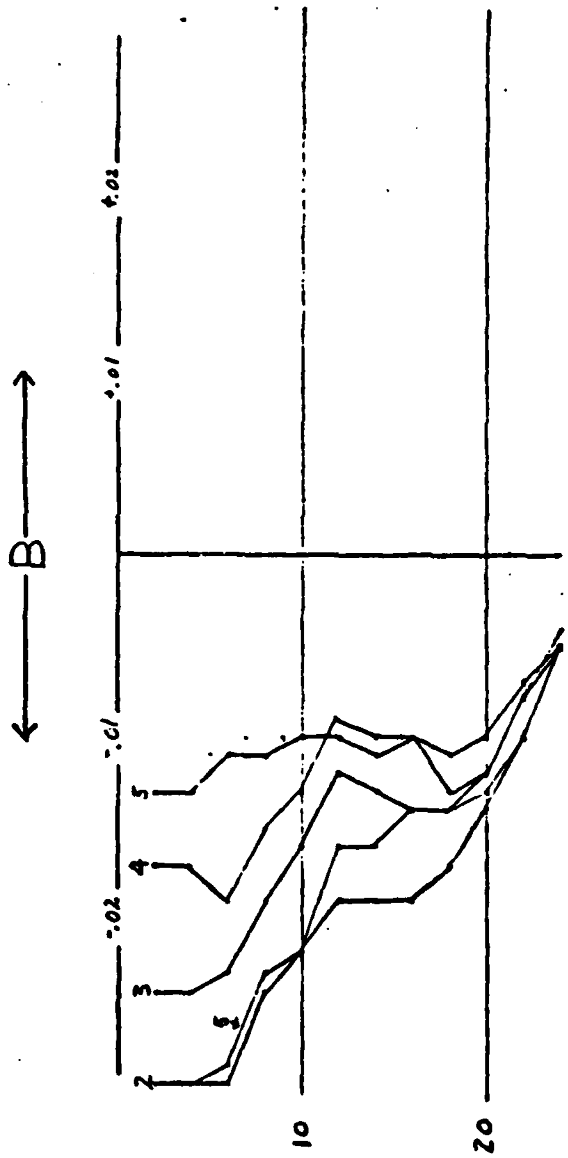
Inclinometer Records

FOR Hole 15 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975



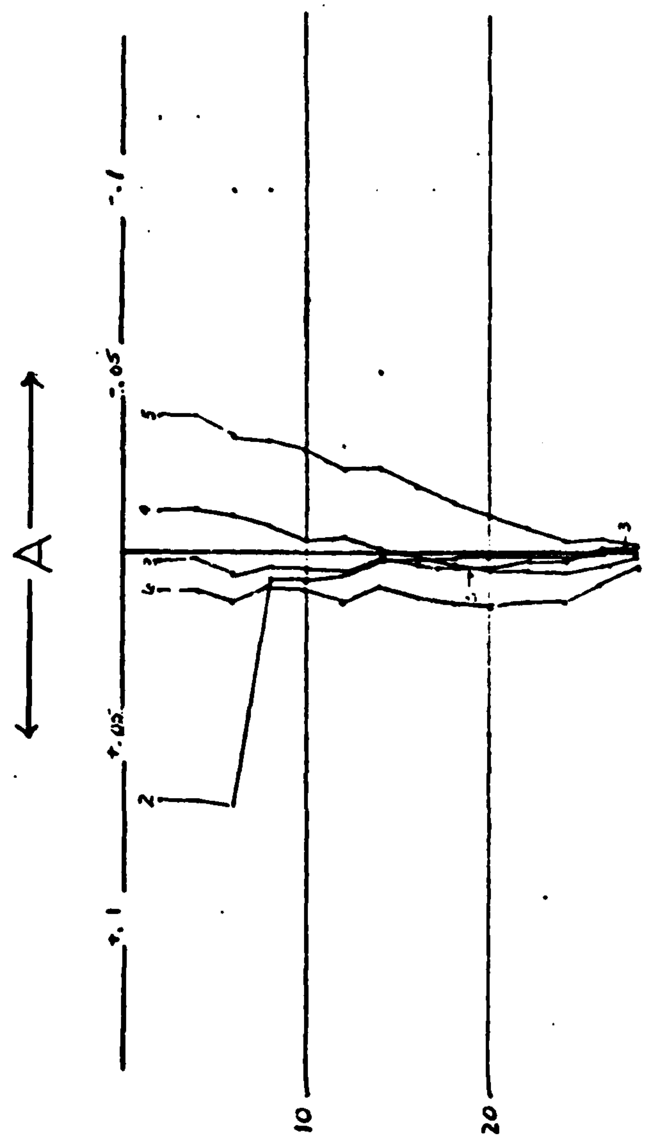
Inclinometer Records

FOR Hole 16 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

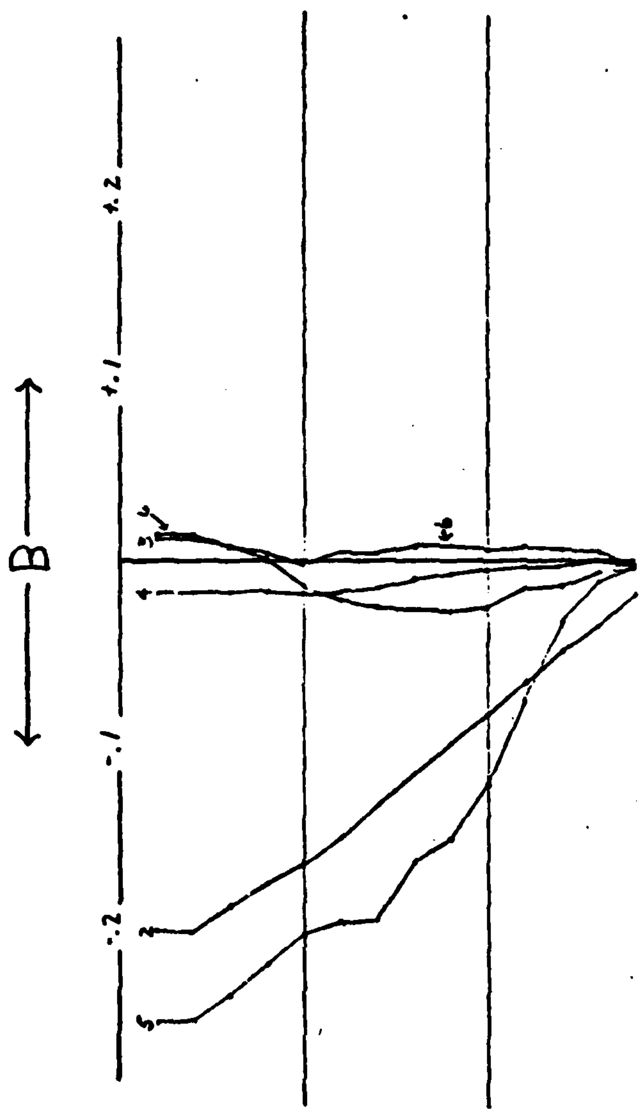
DATE August 23, 1975



Inclinometer Records
FOR Hole 16 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090
DATE August 23, 1975



PROJECT

Inclinometer Records

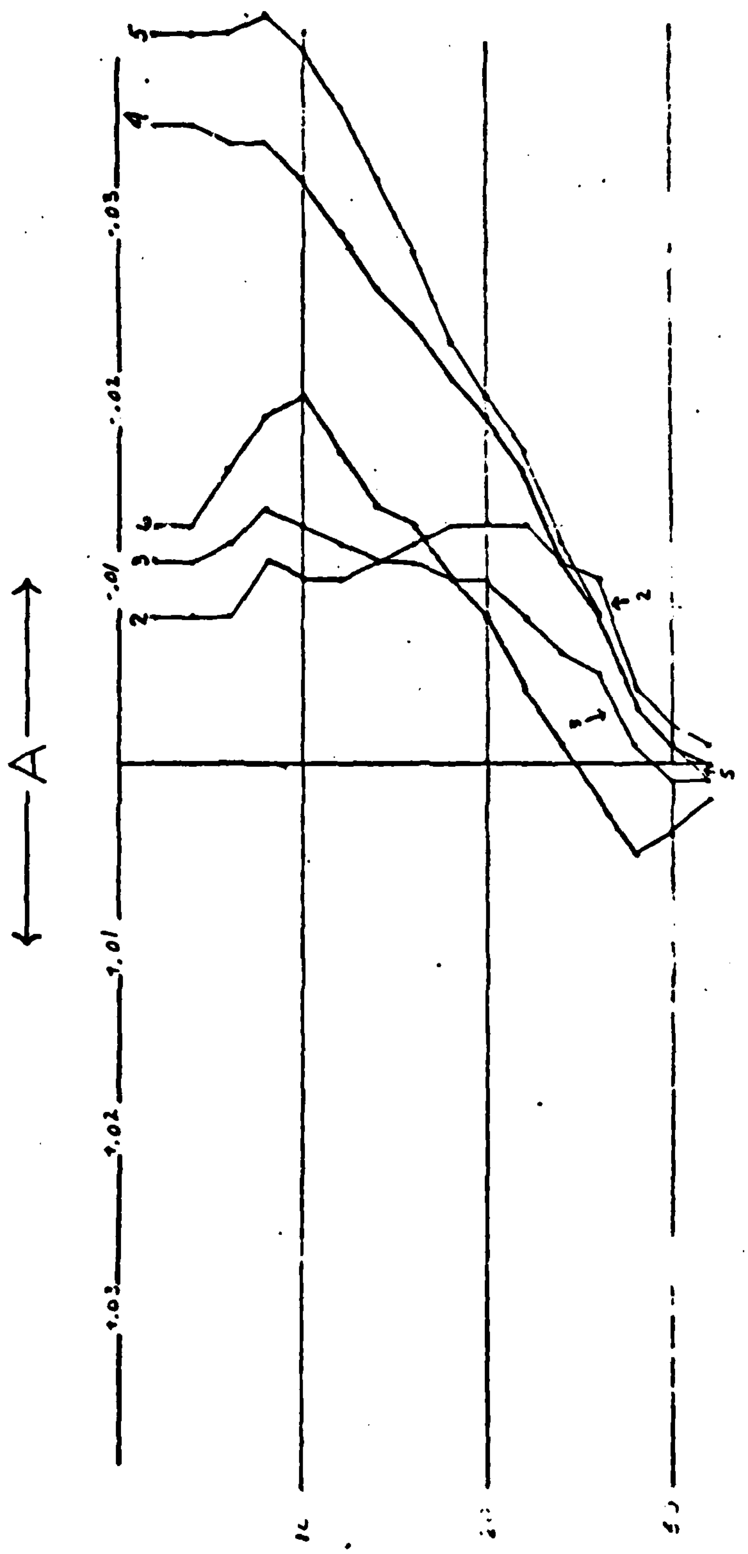
FOR Hole 17 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO. OF

PROJECT NO. 75-090

DATE August 23, 1975



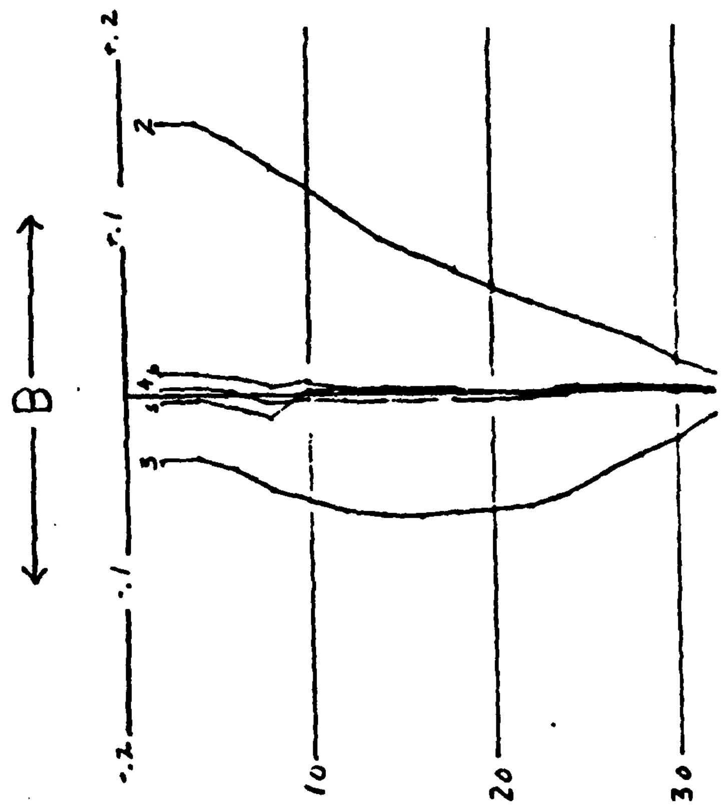
Inclinometer Records

FOR Hole 17 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975

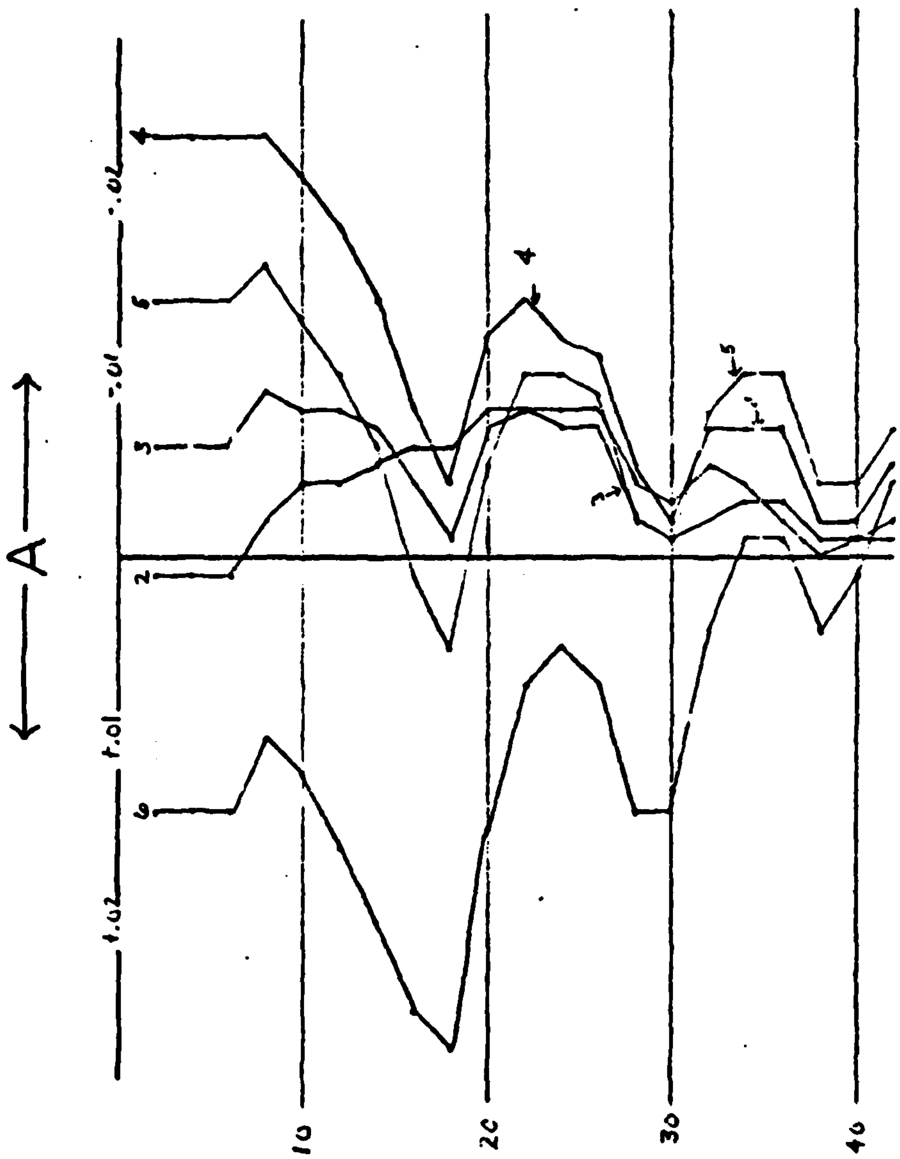


Inclinometer Records

FOR Hole 18 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090
DATE August 23, 1975



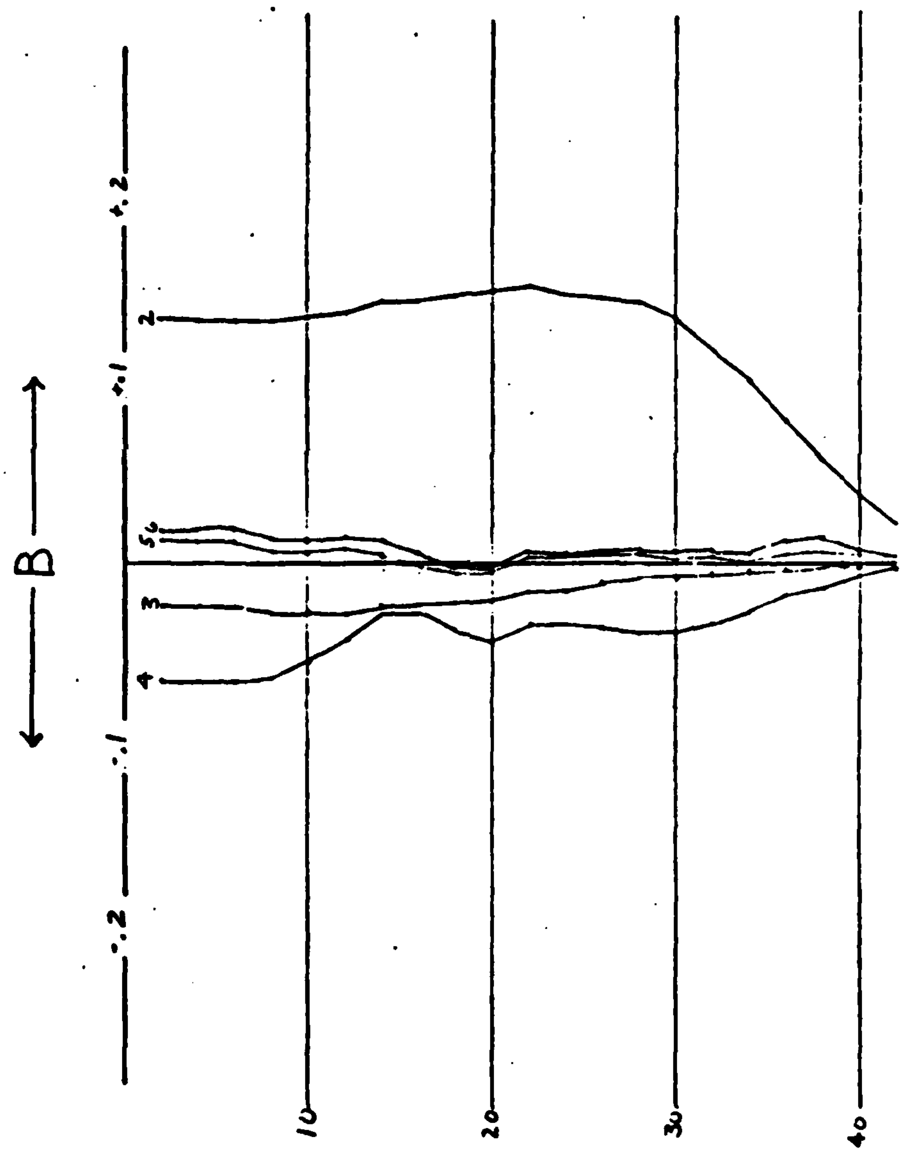
Inclinometer Records

FOR Hole 18 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

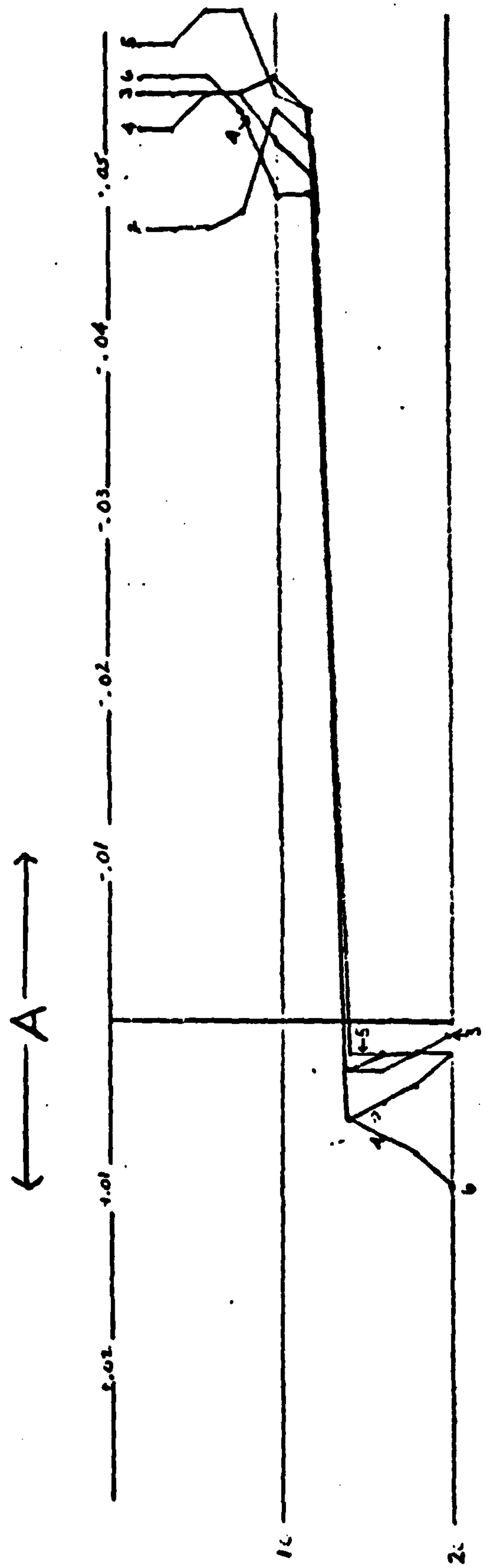
DATE August 23, 1975



PROJECT 8th, Avenue Reservoir
Inclinometer Records
FOR Hole 19 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO. 19 OF
PROJECT NO. 75-090
DATE August 23, 1975



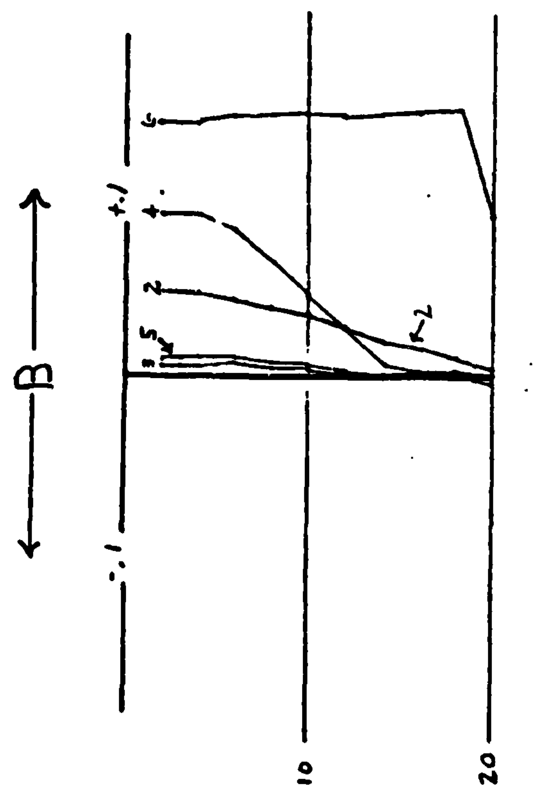
Inclinometer Records

FOR Hole 19 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE August 23, 1975



PROJECT 6th Avenue Reservoir

Inclinometer Records

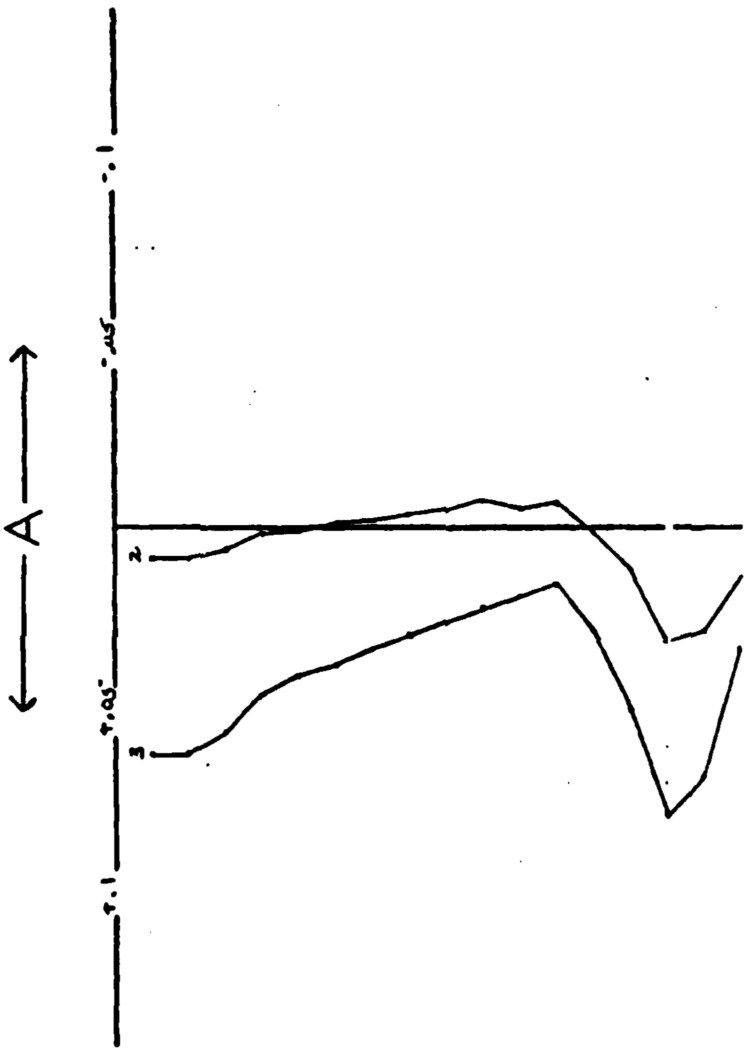
FOR Hole 28 BY

GEOLOGIC ASSOCIATES INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO. [illegible] OF

PROJECT NO. 75-090

DATE October 9, 1975



PROJECT Inclinometer Records

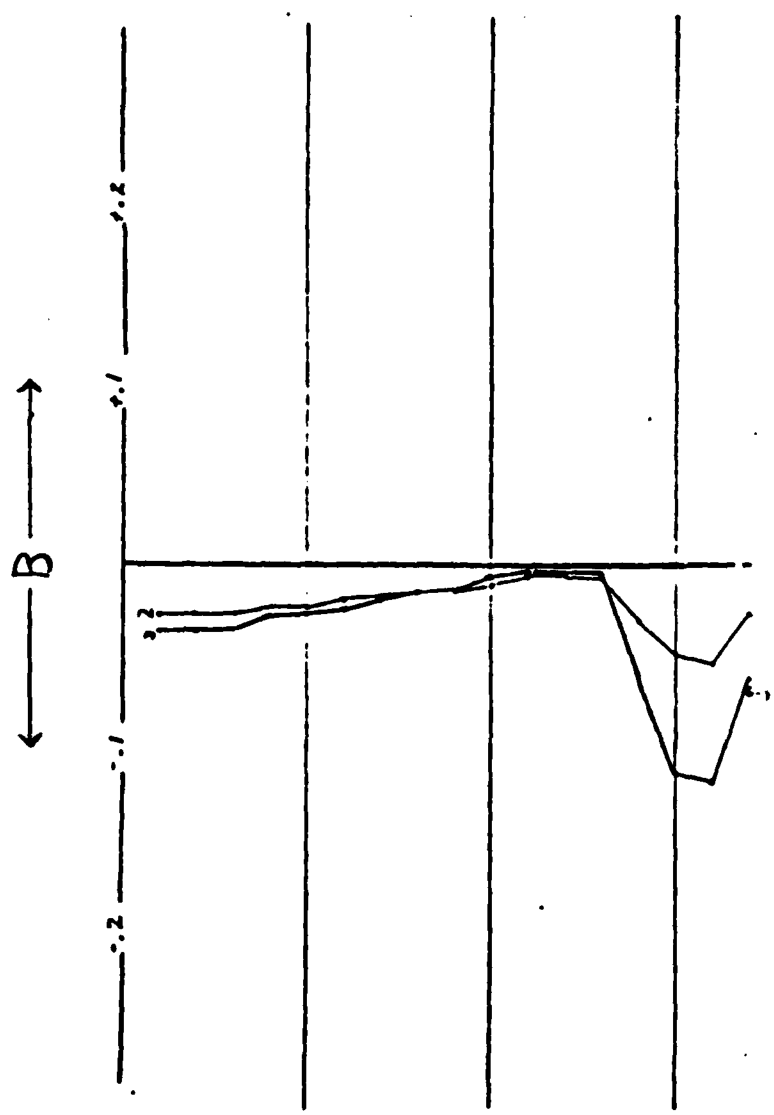
FOR Hole 28 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO. ___ OF ___

PROJECT NO. 75-090

DATE October 9, 1975



PROJECT Inclinometer Records

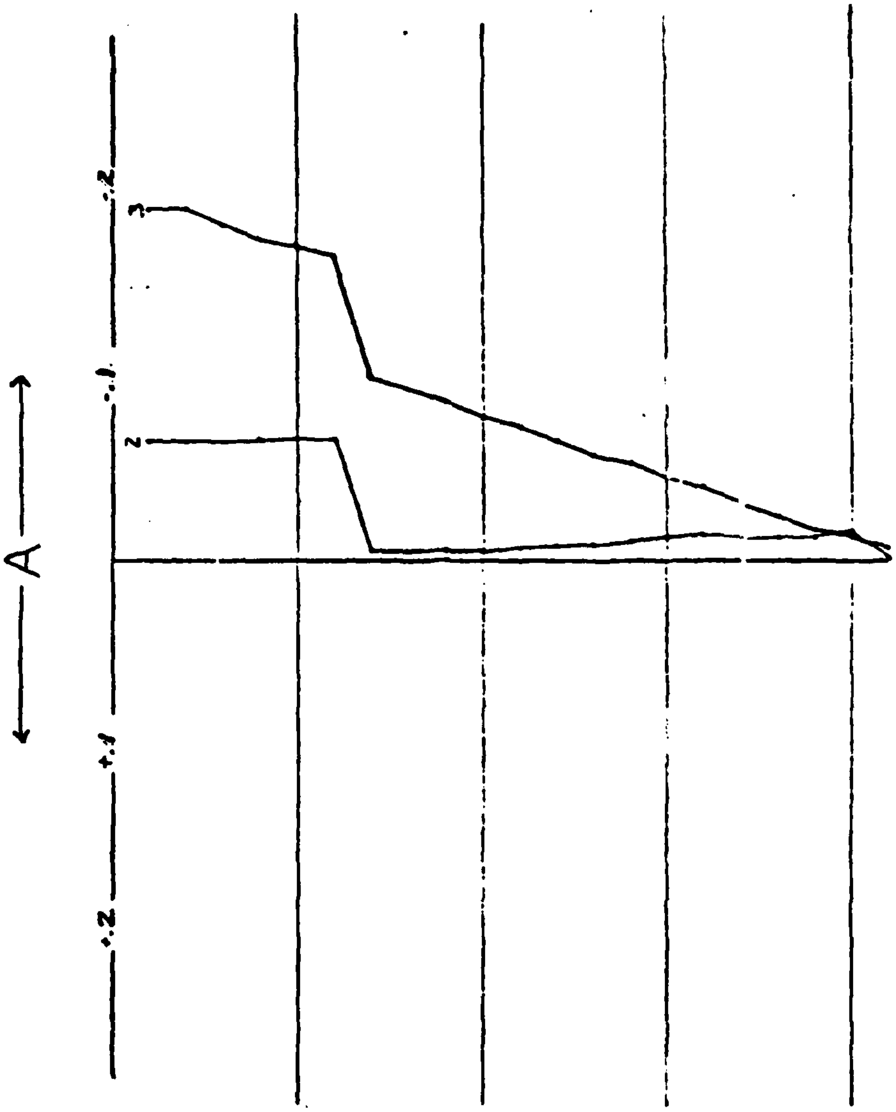
FOR Hole 29 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO.

PROJECT NO. 75-090

DATE October 9, 1975



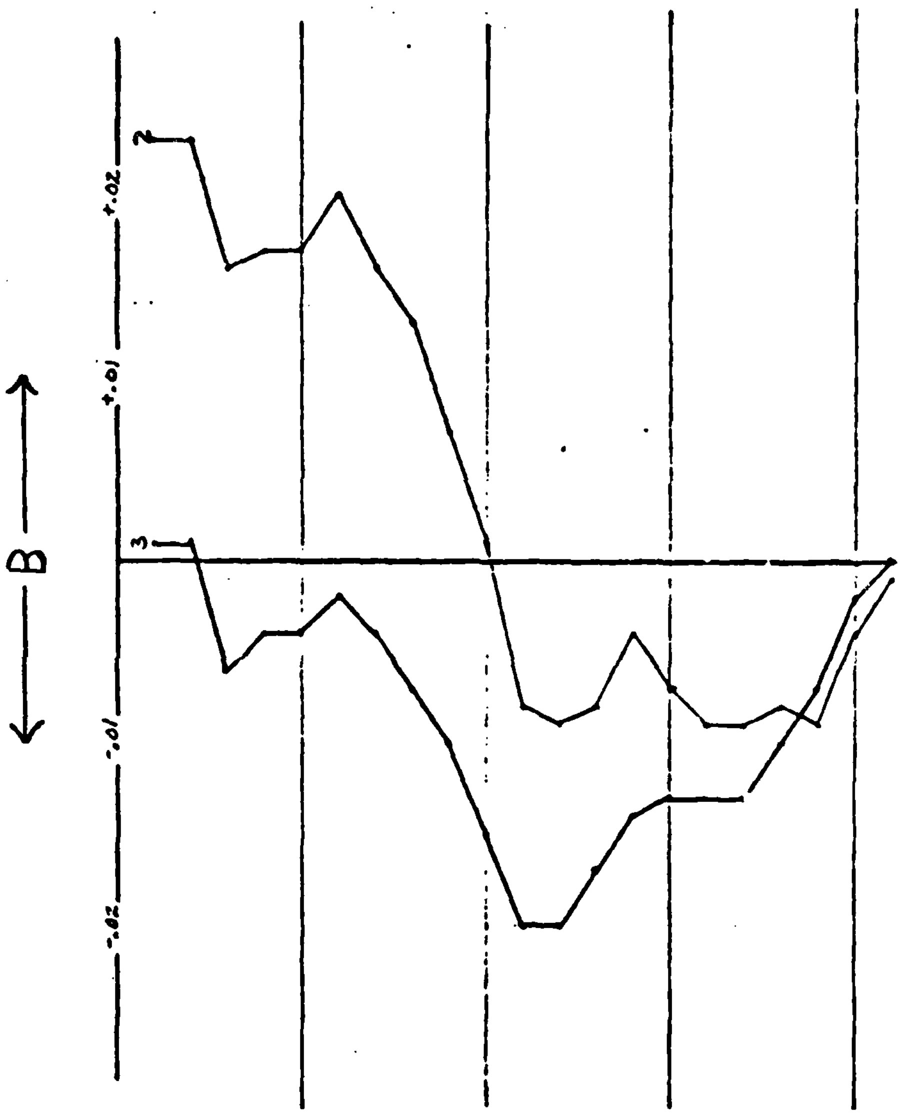
Inclinometer Records

FOR Hole 29 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE October 9, 1975

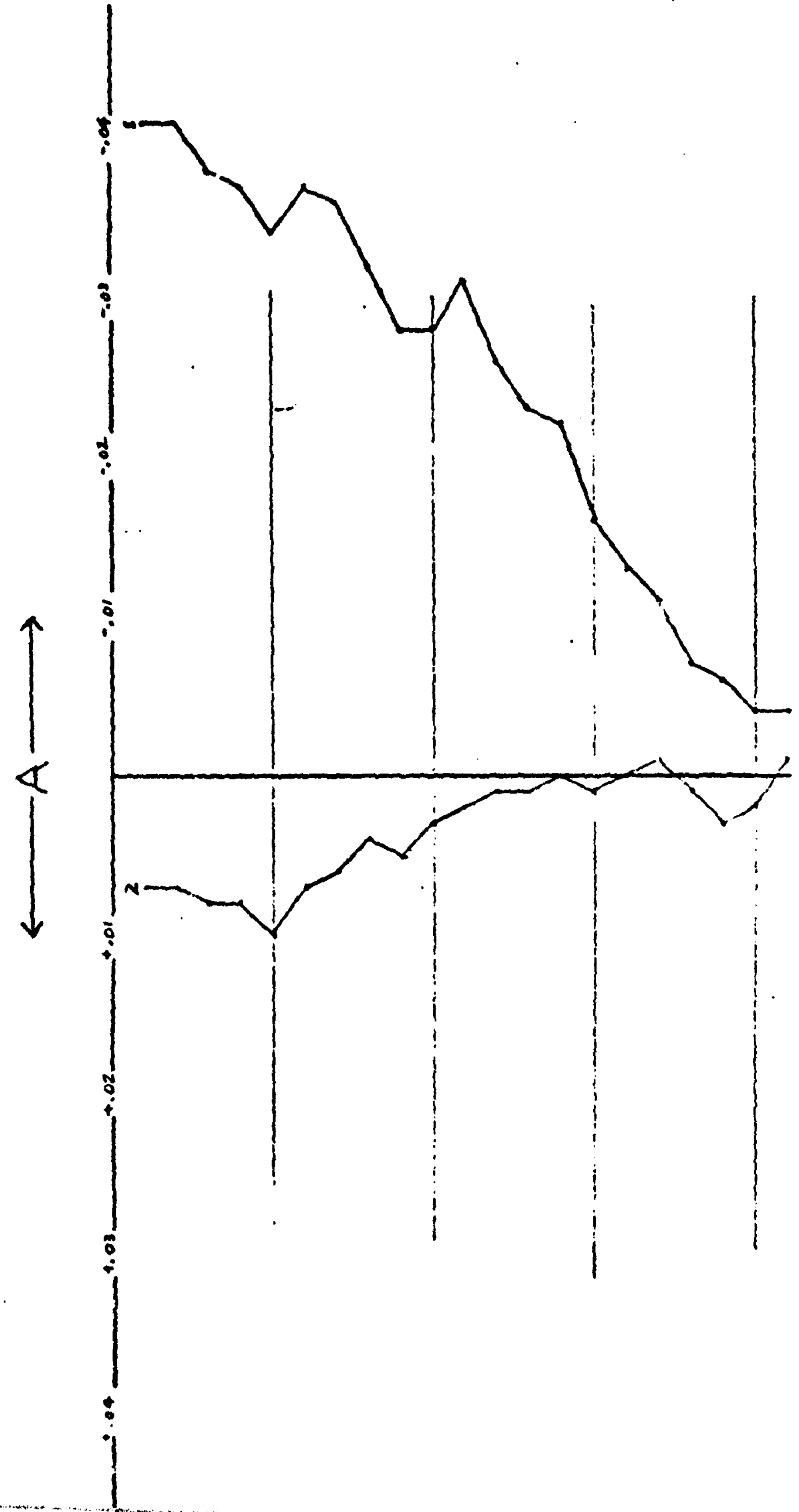


PROJECT Inclinometer Records

FOR Hole 30 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET NO.
PROJECT NO. 75-090
DATE October 9, 1975



PROJECT Inclinometer Records

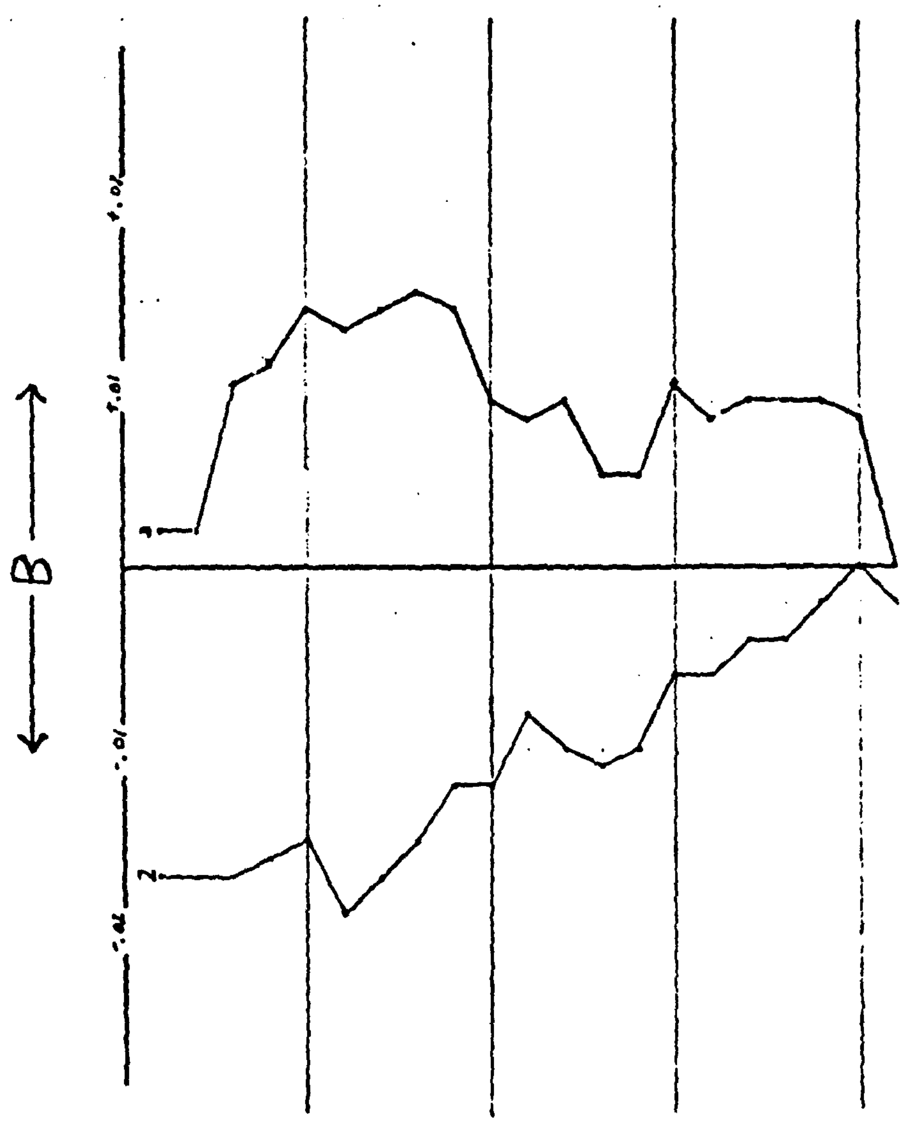
FOR Hole 30 BY

GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

SHEET

PROJECT NO. 75-090

DATE October 9, 1975



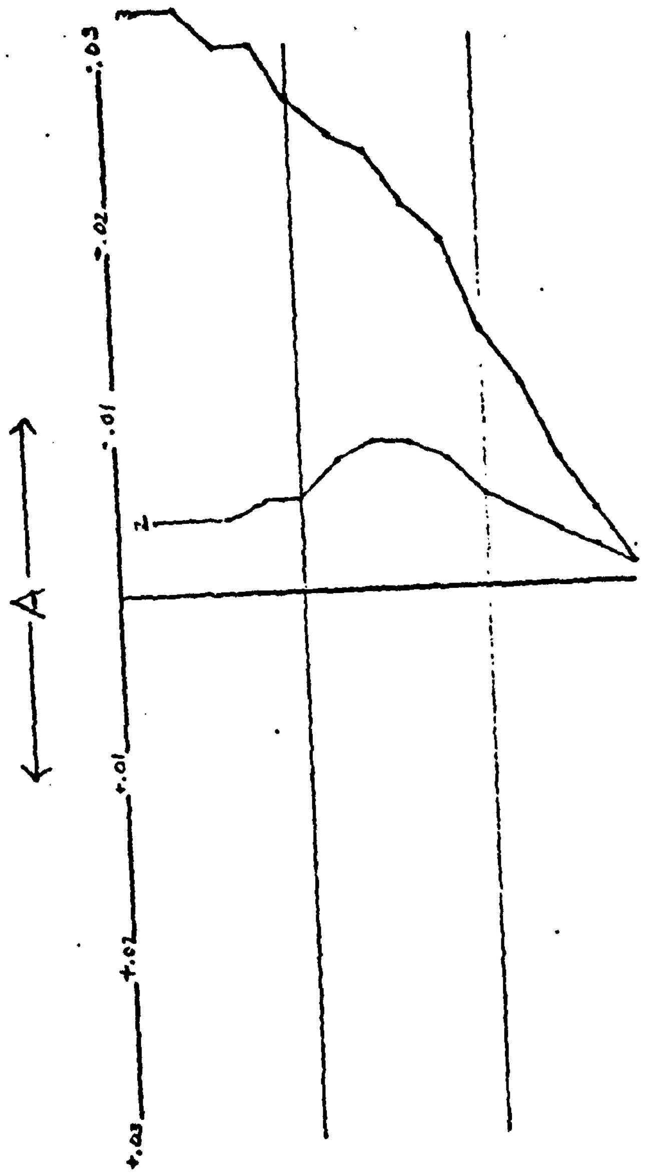
Inclinometer Records

FOR Hole 35 BY

ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

PROJECT NO. 75-090

DATE October 9, 1975



PROJECT Inclinometer Records

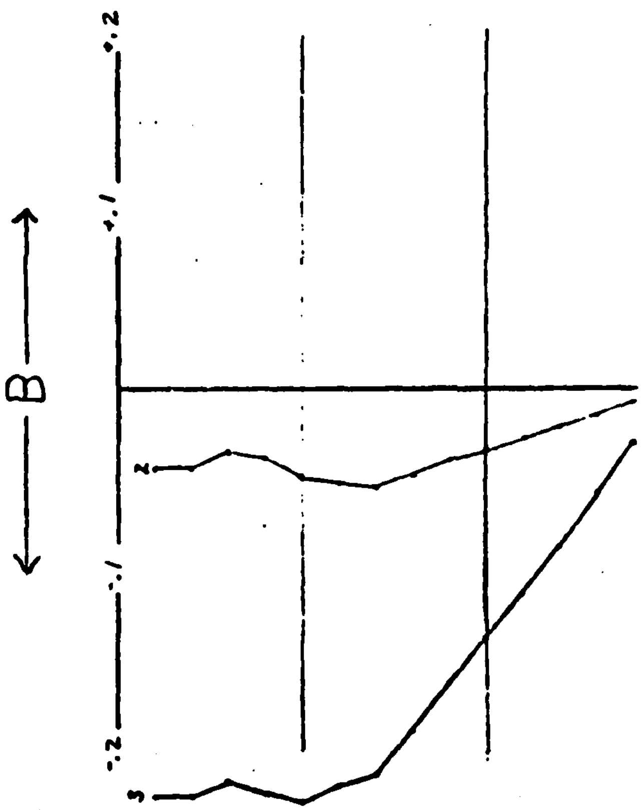
FOR Hole 35 BY

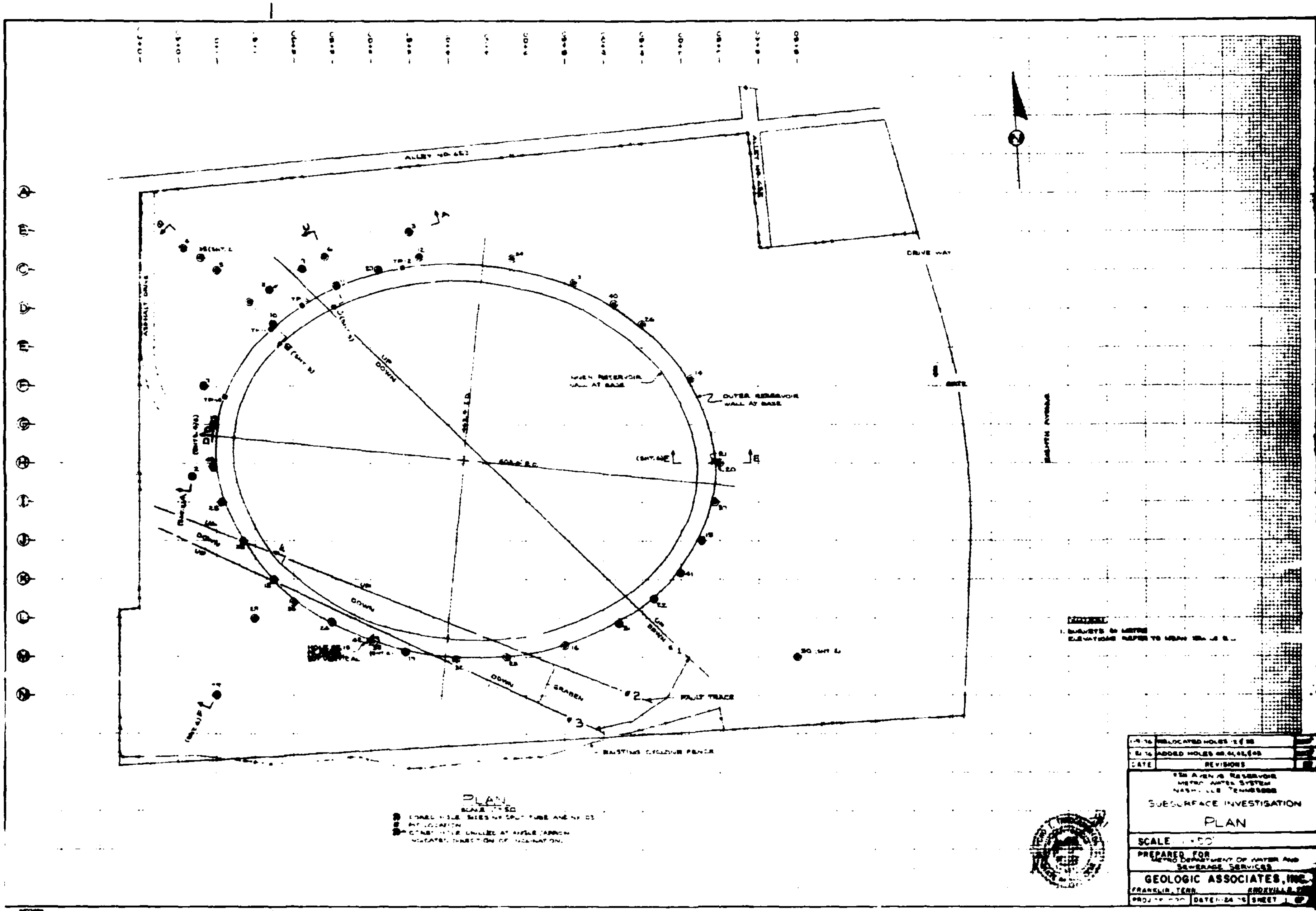
GEOLOGIC ASSOCIATES, INC.
ENGINEERS & GEOLOGISTS
FRANKLIN, TENNESSEE
KNOXVILLE, TENNESSEE

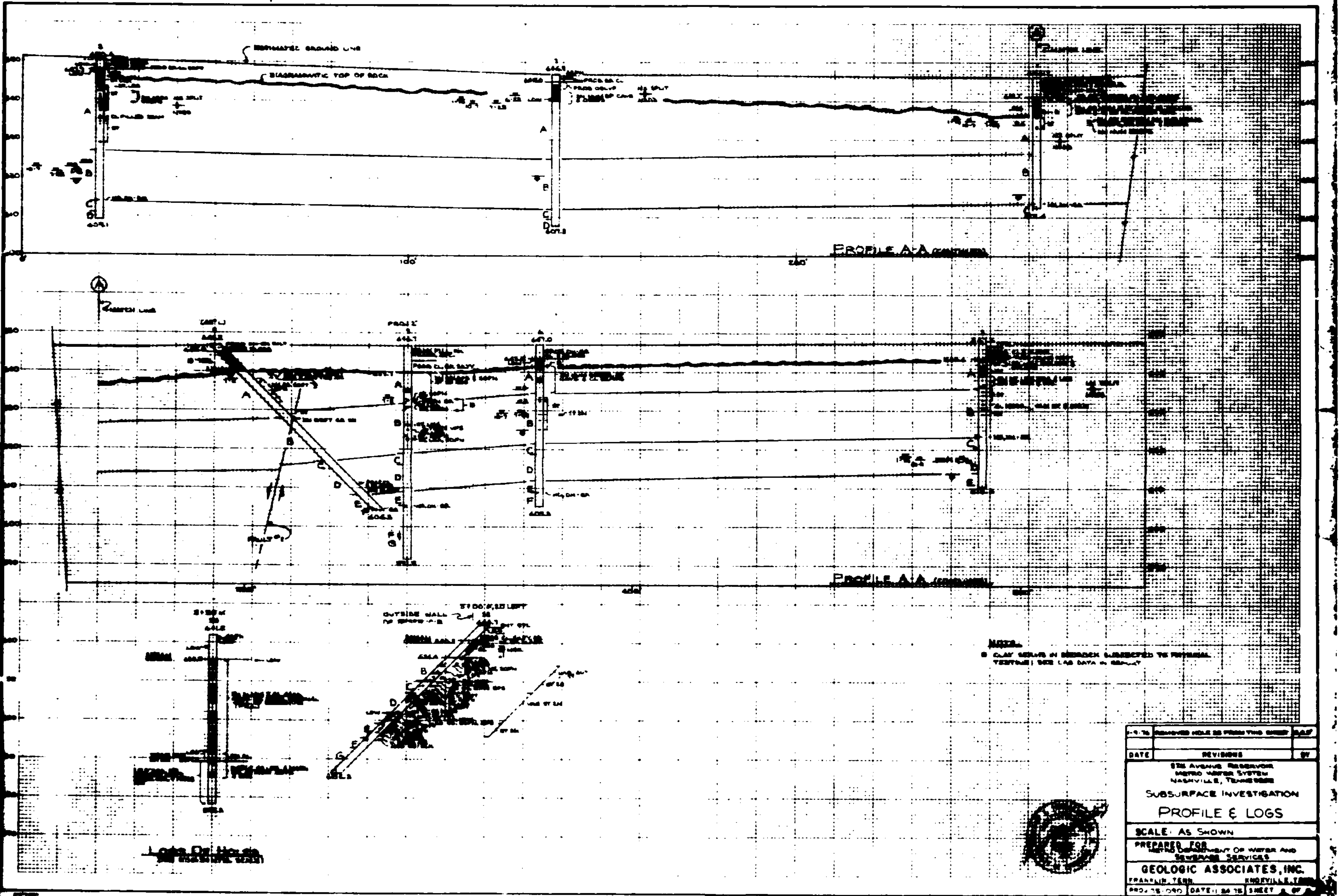
SHEET NO ______ OF ______

PROJECT NO. 75-090

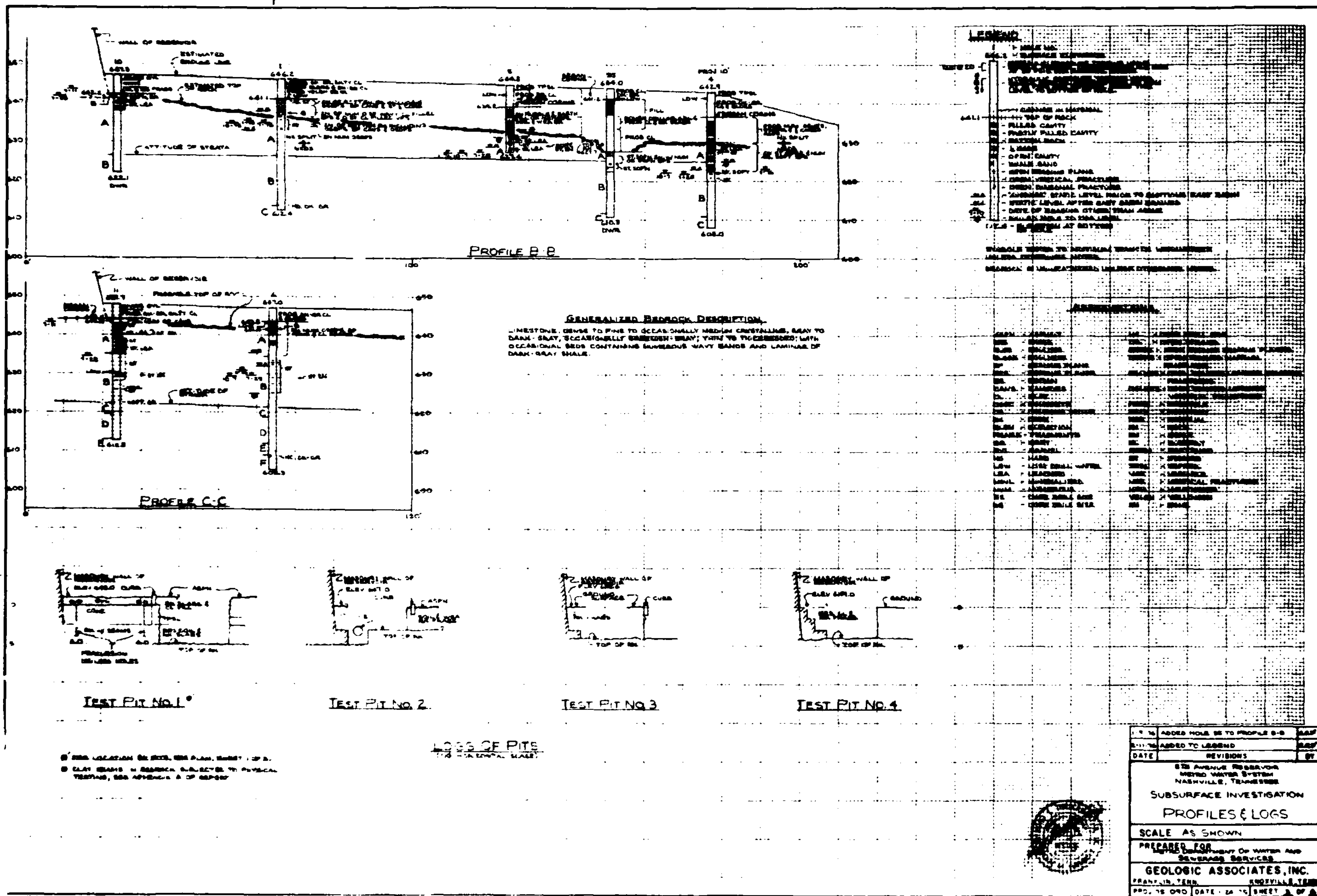
DATE October 9, 1975

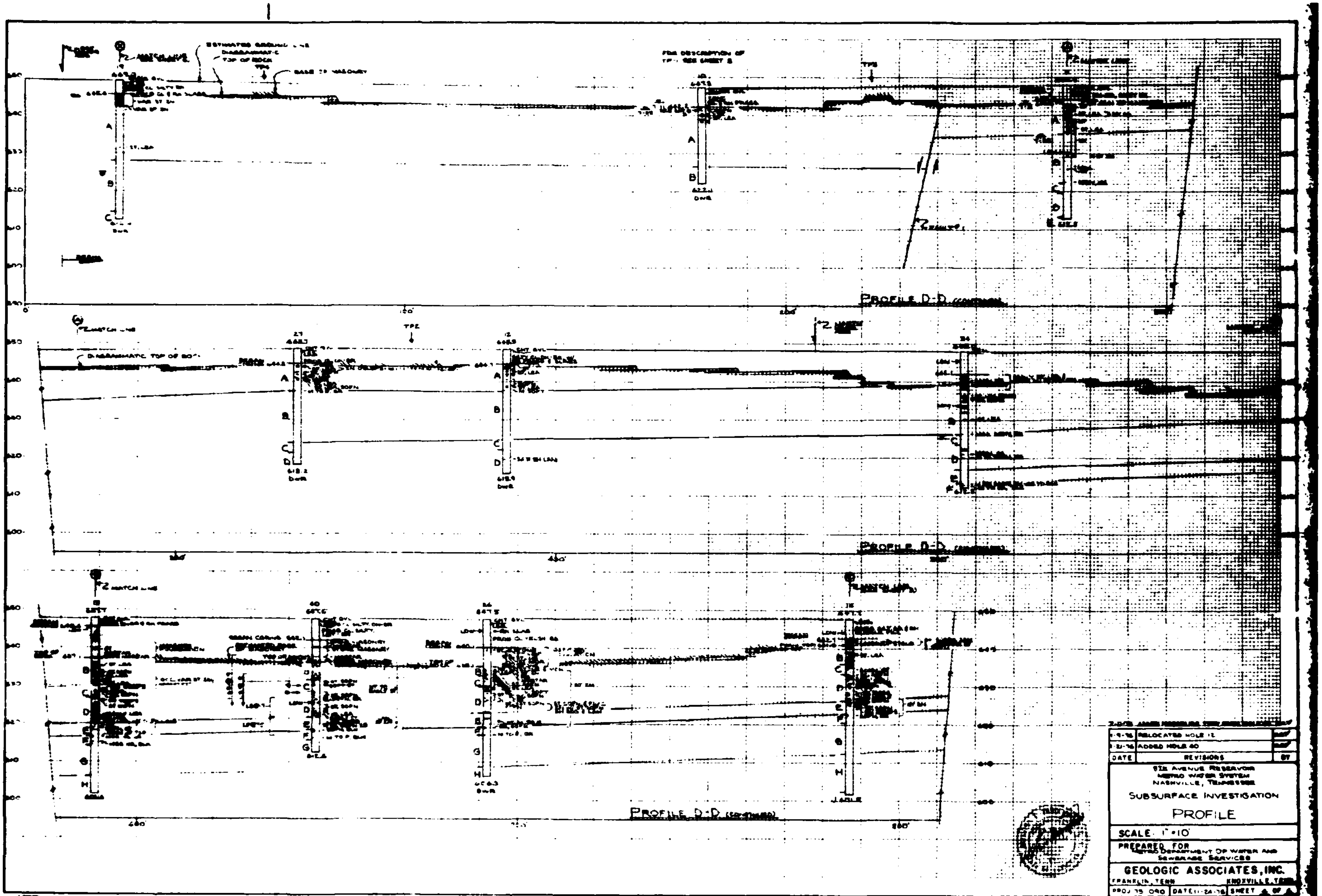


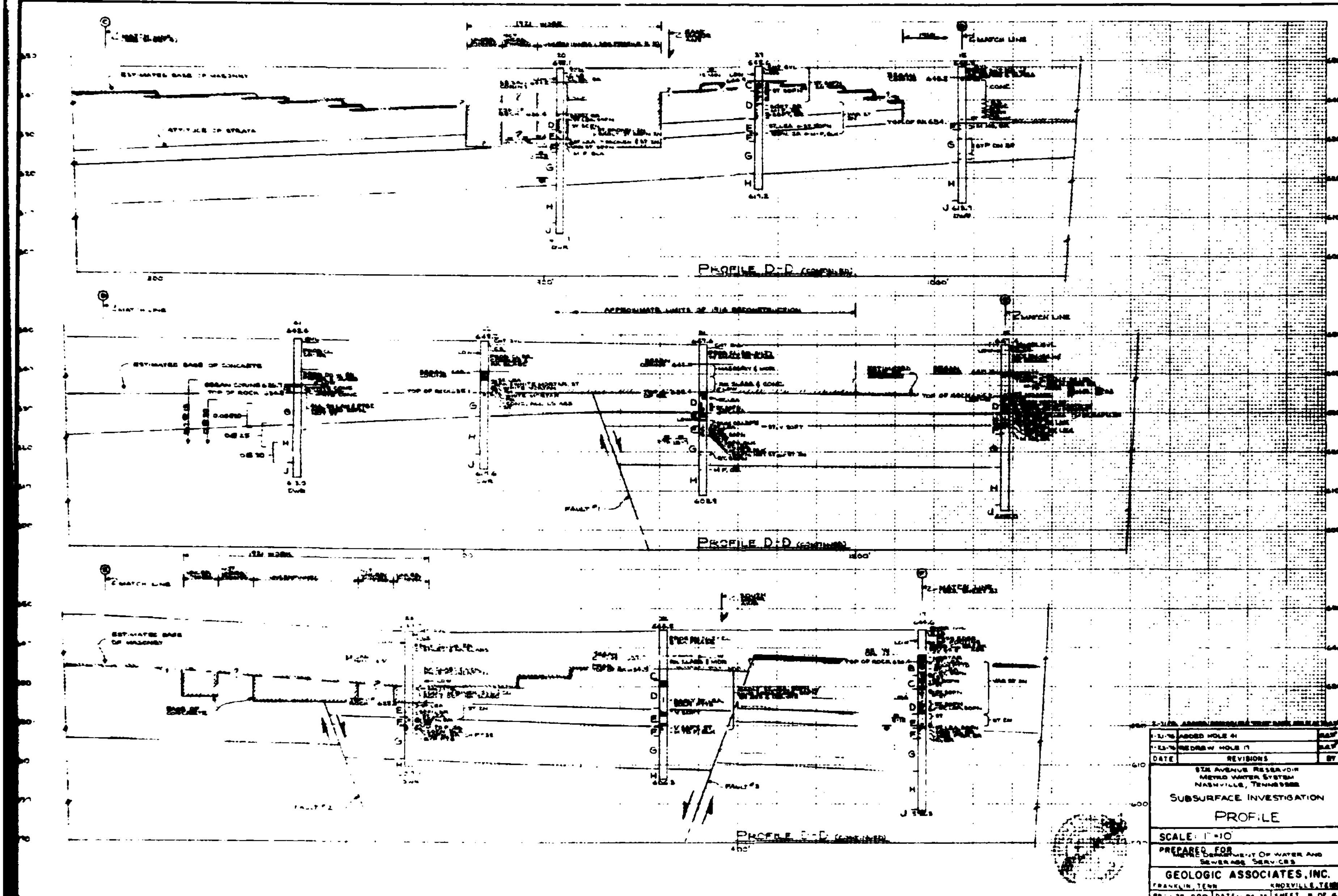

ALLEY NO. 462
DRIVE WAY
GATE
ASPHALT DRIVE
INNER RESERVOIR WALL AT BASE
OUTER RESERVOIR WALL AT BASE
UP
DOWN
GRABEN
FAULT TRACE
EXISTING CYCLONE FENCE
#2
#3
PLAN
REVISIONS
SUBSURFACE INVESTIGATION
PLAN
SCALE
PREPARED FOR
GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN.
DATE
SHEET



PROFILE A-A
PROFILE A-A
DIAGRAMMATIC TOP OF ROCK
OUTSIDE WALL
LOGS OF HOLES
NOTES
SUBSURFACE INVESTIGATION
PROFILE & LOGS
SCALE: AS SHOWN
PREPARED FOR
GEOLOGIC ASSOCIATES, INC.


LEGEND
PROFILE B-B
PROFILE C-C
GENERALIZED BEDROCK DESCRIPTION
LIMESTONE, DENSE TO FINE TO OCCASIONALLY MEDIUM CRYSTALLINE, GRAY TO DARK-GRAY, OCCASIONALLY BROWNISH-GRAY; THIN TO THICK BEDDED; WITH OCCASIONAL BEDS CONTAINING NUMEROUS WAVY BANDS AND LAMINAE OF DARK-GRAY SHALE.
TEST PIT NO. 1
TEST PIT NO. 2
TEST PIT NO. 3
TEST PIT NO. 4
LOGS OF PITS
ADDED TO LEGEND
DATE
REVISIONS
BY
NASHVILLE, TENNESSEE
SUBSURFACE INVESTIGATION
PROFILES & LOGS
SCALE AS SHOWN
PREPARED FOR
SEWERAGE SERVICES
GEOLOGIC ASSOCIATES, INC.
SHEET


PROFILE D-D (CONTINUED)
PROFILE D-D (CONTINUED)
PROFILE D-D (CONTINUED)
ESTIMATED GROUND LINE
DIAGRAMMATIC TOP OF ROCK
BASE OF MASONRY
DIAGRAMMATIC TOP OF ROCK
RELOCATED HOLE 12
ADDED HOLE 40
DATE
REVISIONS
BY
SUBSURFACE INVESTIGATION
PROFILE
SCALE: 1" = 10'
PREPARED FOR
GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN
KNOXVILLE, TENN



DATE	REVISIONS	BY
1-21-76	ADDED HOLE 41	
1-23-76	REDRAW HOLE 17	

8TH AVENUE RESERVOIR
METRO WATER SYSTEM
NASHVILLE, TENNESSEE

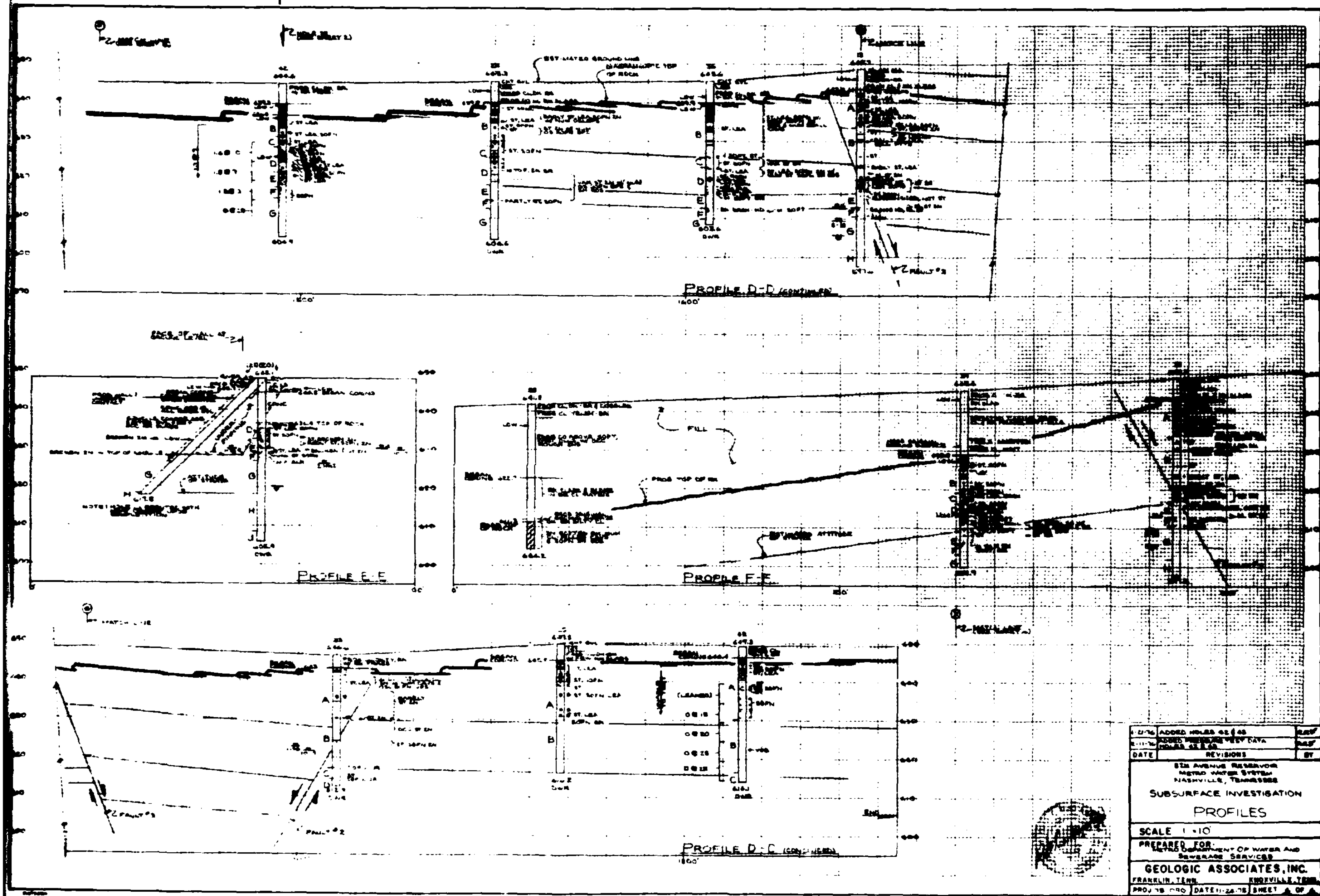
SUBSURFACE INVESTIGATION

PROFILE

SCALE: 1" = 10'

PREPARED FOR
METRO DEPARTMENT OF WATER AND SEWERAGE SERVICES

GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN. KNOXVILLE, TENN.



PROFILE D-D (CONTINUED)
PROFILE E-E
PROFILE F-F
PROFILE D-C (CONTINUED)
FILL
ADDED HOLES 42 & 43
REVISIONS
8TH AVENUE RESERVOIR
METRO WATER SYSTEM
NASHVILLE, TENNESSEE
SUBSURFACE INVESTIGATION
PROFILES
SCALE 1"=10'
PREPARED FOR: METRO DEPARTMENT OF WATER AND SEWERAGE SERVICES
GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN.
KNOXVILLE, TENN.

APPENDIX B

Review of Stability

Eight Avenue Reservoir

Nashville, Tennessee

January 1976

Shannon & Wilson, Inc.

Review of Stability
Eighth Avenue Reservoir
Nashville, Tennessee

For

Geologic Associates, Inc.

January, 1976

Shannon & Wilson, Inc.
Geotechnical Consultants
Burlingame, California

TABLE OF CONTENTS

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LIST OF FIGURES

REVIEW OF STABILITY
EIGHTH AVENUE RESERVOIR
NASHVILLE, TENNESSEE

Purpose and Scope

The principal purpose of our studies is to determine if it is feasible to instrument the reservoir structure and foundation to provide advance warning of movement which might precede a failure. If feasible, the type(s) of suitable instrumentation should be defined.

The scope of study includes an inspection of the reservoir and discussions with Messrs. Ray Throckmorton and Ed Wilson of Geologic Associates and their consultant, Mr. William Gardner, Consulting Geologist. In addition, we have reviewed:

1. Report of Subsurface Investigation, Eighth Avenue Reservoir, Nashville, Tennessee, Volumes I and II dated November 25, 1975, by Geologic Associates, Inc.

2. Plans of borings and geologic profiles by Geologic Associates.

3. Rock core from representative borings.

4. Reconstructed photographs of the 1912 failure.

As a result of site inspection and subsequent discussions, we were also requested to comment on the general stability of the reservoir, the need for permanent repairs, and methods of effecting the repairs.

This report summarizes and amplifies our discussions.

Background

The reservoir was constructed in 1889; it is elliptical in shape with a long axis of 603 feet, a short axis of 463 feet, and a circumference of about 1,800 feet. The gravity type walls are constructed of a cut stone masonry facing with an interior filling of cyclopean concrete. The height of the wall above the basin floor is about 33 feet. The thickness of the top of the wall is eight feet, and the base width at the floor of the reservoir is about 23 feet. Both faces of the wall have a concave shape.

In 1912, a 200-foot section of the wall in the southeast quadrant failed. The mechanism of failure, as deduced from a few photographs and current information on foundation conditions, appears to have been horizontal sliding on weak shale or clay layers within the foundation rock. Uplift pressures acting in the foundation and resulting from leakage from the reservoir may also have been an important factor, but no information on leakage or foundation water pressures are currently available.

The failed portion of the wall was reconstructed in 1914. In addition, a section of the wall in the northwest quadrant was investigated by test pits and apparently buttressed with concrete in one location. The inside of the wall and floor of the basin was also lined with gunite.

Distress was noticed in the wall in 1920; and after draining the east basin, a fine (1/64 inch wide) horizontal crack located about six feet above the basin floor and extending 120 feet northerly from the rebuilt section was observed. Some vertical cracking in the outside face of the masonry was also noticed. The problem was investigated, and as a result, portions of the wall to the north and south of the reconstructed section were underpinned, and the walls of the entire reservoir were scaled and regunited. Another concrete floor was placed over the existing floor.

After cleaning the west basin in late 1974 and resuming operation in March 1975, increased leakage was noticed in the above grade portion of the west wall. As a result, foundation investigations were initiated by Geologic Associates. These investigations included core borings, ground water level observations, installation and monitoring of inclinometer casings, test pits, and the installation of precise tilt meters on the crest of the wall at four locations. Concern about the stability of the wall led to draining the west basin in August 1975, and it presently remains out of service.

Summary of Present Conditions

The reservoir structure is generally founded on predominantly flat-lying, shaley limestone with interbeds of shale. In the proximity of the surface, the shales are often weathered to clay. The depth of weathering varies widely and appears to be related to intensity and location of jointing and possibly faulting. The strength of the shale and clay strata control the strength of the foundation and are not well known.

Underlying the weathered zone is fresh rock also composed of predominantly shaley limestone with interbeds of shale.

The shallow foundation rock appears to be relatively permeable due to jointing and degree of weathering. The tightness of the formation increases with depth.

The observation wells installed in borings indicate the presence of water in the foundation rock. The water apparently is derived from reservoir leakage and is significantly higher along the west basin. A drop in water level was noted in adjacent borings when the west basin was drained. No information is currently available on the amount of reservoir leakage.

Flowing leaks through the masonry wall of the west basin were observed in March 1975. In January 1976, approximately four

months after the basin was drained, moss and lichens were observed 10 to 15 feet above the exterior grade. It has been reported that the flow through the visible portion of the wall increases when exposed to the sun and decreases when in the shade and at night.

The failure in 1912 apparently occurred along flat-lying weak shale or clay strata in the foundation. Uplift pressures due to leakage may also have been an important factor. A degradation of the strength of the weak shale or clay layers due to the presence of leakage water, an increase in uplift pressure due to increased leakage, or a combination of these factors may have initiated the 1912 failure.

Inclinometer casings were installed in the 1975 investigations and have been observed periodically. These data show small variations which are attributed to normal scatter of the data within the range of accuracy of the instrument. In our opinion, no consistent foundation movement trends are indicated by the data.

The relatively poor foundation conditions disclosed by the borings combined with the prior history of the structure indicate that the structure is of marginal stability and has a low reserve of safety against foundation sliding. A review of the original stability analysis which did not consider uplift forces and recent calculations also support this conclusion.

Although the structure is located in an area not expected to be affected by severe seismic ground shaking, the condition of the foundation suggests that the structure and its water-tight integrity could be affected adversely by even very modest shaking. Minor adjustments of the structure or its lining due to shaking could initiate changes in hydrostatic conditions in the foundation which could lead to failure.

In our opinion, if the reservoir is to be used for an extended period, steps should be taken at an early date to permanently strengthen the structure and foundation in order to increase the reserve of safety against foundation failure to presently acceptable standards.

We did not observe the leakage through the west basin wall, but the reported leakage appears excessive. Steps should also be taken to eliminate or reduce leakage through the exterior walls and the foundation to acceptable levels. Continued leakage at the present rate would be detrimental to both the masonry wall and the weathered rock foundation.

Although we are not qualified to render an opinion on the condition of the masonry wall, it appears that its structural condition should be investigated by drilling and coring. These investigations would also lead to an improved understanding of the water levels within the masonry wall.

It is further our opinion that if the west basin is to be filled and both basins operated for an extended period in its present condition prior to permanent repairs, it should be instrumented to provide an early indication of small foundation movements. Small detectable movements would be expected to precede any large-scale movements and can be used as an advance indication of imminent failure.

The instrumentation system should also be installed at an early date to obtain a history of performance of the filled east basin and of the empty west basin. Furthermore, the west basin should be filled in a controlled manner with frequent observations of the instruments to evaluate foundation performance. If excessive deformations develop or if deformations persist with time, it may be necessary to stop the filling of the basin.

Instrumentation Systems

The hypothesized method of failure is sliding along flat lying, thin shale or clay beds within the weathered zone of the foundation rock. The amount of progressive foundation deformation prior to failure would be expected to be small and probably in the order of a fraction of an inch. Therefore, the intrumentation system must be capable of reliably measuring very small movements of the structure and foundation in order to resolve trends of deformation at an early stage. Furthermore, the system should be permanently installed and wired to automatically provide an alarm if preset limits of movement are exceeded.

In our opinion, a rod-type extensometer system may be used to make the required measurements. Eighteen instruments should be spaced at about 90-foot centers as shown in Fig. 1 on the premise that any significant movements would involve more than 90 feet of wall and thus would affect one or more instruments. The extensometers are shown in section on Fig. 2 and 3, respectively, for the shallowest and deepest sound rock. The extensometers spacing may be increased in the reconstructed and underpinned areas located in the southeast quadrant of the wall.

The elements of the system would include the following:

1. Percussion or core drilled, two-inch diameter borings to sound rock at an angle of 30 degrees to the vertical.

2. Stainless steel, 0.25-inch tubing encased in 0.5-inch PVC tubing and fitted with a hydraulically expanded anchor. The anchor would be set in sound rock at the base of the hole and the whole assembly grouted. Four rod assemblies would be made of invar to accurately measure movements related to temperature changes in the rod.

3. A linear potentiometer sensor with a sensitivity of 0.002 inch. The travel would be ±1.0 inch.

4. Terminal sensor housing attached to the face of the wall or embedded in the wall.

5. Underground connecting cable leading to a central recording station.

6. Central control terminal with automatic low voltage readout circuitry.

The low voltage readout circuitry would continuously scan the output of each extensometer sensor in sequence, compare the changes to preset limits, and automatically activate an alarm if the limits are exceeded. The system would also print out the data from each sensor automatically upon demand.

The preset limits would be determined by experience after the first extensometers are installed. If desired, an advance installation of a few instruments may be made to establish the movement characteristics of the wall, before a large-scale installation is made. Such an experimental installation would not incorporate automatic recording or alarm systems.

The approximate costs of the materials and equipment for the 18-unit system are shown on Table 1. These costs exclude drilling, grouting, and labor for installation.

In addition to measuring foundation deformations, it would also be desirable to collect and monitor seepage on a long-term basis as an indication of changing conditions and possibly more severe problems. However, we have studied a subdrain system around the perimeter of the basin, and it appears that such a system would have to be installed well below the top of rock to

intercept any significant amount of seepage. The difficulty and cost of such a system does not appear justified.

Permanent Repairs

A number of schemes have been suggested for consideration, although to date, none have been studied in detail. The following comments apply to each scheme.

Pressure grouting. This scheme would involve the injection of cement grout using a sequence of vertical and inclined holes along one face of the wall. Presumably the grout would strengthen the foundation and reduce leakage.

In our opinion, a grouting program should only be undertaken on the inside face of the structure and should be accompanied by installation of a drainage system beneath and/or along the outside edge of the wall. We also question whether grouting will significantly improve the strength of the foundation. If relatively high uplift pressures are present beneath the structure then grouting and the associated drainage measures may reduce uplift pressures and improve stability. If high uplift pressures are not present beneath the structure, then grouting would probably not greatly improve stability. It is also possible that sealing the foundation may also result in a net increase in the horizontal thrust due to hydrostatic pressure and, therefore, decrease the reserve of stability of the structure.

If grouting is planned, more detailed information should be obtained regarding existing uplift pressures beneath the structure and the groutability of the formation.

Waterproofing. It is clear that positive steps must be taken to eliminate or reduce seepage through the masonry wall to acceptable limits. Such seepage will cause progressive deterioration of the structure and drastically reduce its life span.

Limited success has been achieved with relatively rigid gunite linings in the past. The effectiveness of the gunite has undoubtedly been affected by the deformations of the wall due to temperature variations and the imposition of the hydrostatic load on the wall. It appears that flexible membrane systems should be considered at this time.

It would also be beneficial to eliminate or reduce foundation leakage. This also may be accomplished with a membrane system. Other types of linings may also be feasible.

Although reductions of reservoir leakage would be beneficial; the effect on foundation stability is not clear. If uplift pressure is not a significant factor, then reduction of leakage may not significantly improve the short-term stability of the system. The reduction of leakage would reduce the rate of weathering of the foundation rock and thus benefit the long-term stability.

Underpin wall. Portions of the wall were underpinned in 1921 by hand mining the defective rock in sections beneath the wall and replacing the rock with concrete. It is clear that this procedure would be effective in adequately increasing the stability of the wall. The obvious disadvantage is the time required to accomplish this method of repair and the present cost of using hand methods. Blasting beneath or adjacent to the existing structure should not be permitted.

Drilled piers. This scheme would consist of a system of piers sustaining a post-tensioned "ring." In our opinion, such a concept can be used to provide the required increase in stability. Consideration may also be given to providing a tension ring without pier support or a system of piers with a nominal cap beam. Due to the potential mechanism of failure along a thin clay layer, the primary mode of resistance of the piers would appear to be shear rather than cantilever bending, but this re-

quires further study before adopting this criterion for design.

Major reconstruction. This scheme would consist of enlarging the reservoir capacity by lowering the floor to sound rock and underpinning the walls. From a technical point of view, this would be possible, but it may not be economically feasible.

Tied-back ring beam. In our opinion, it would be feasible to provide the required increase in stability by constructing a circumferencial girth beam around the base of the structure and anchoring the beam to sound rock with inclined tendon anchors. This scheme is illustrated in section in Fig. 4 and 5.

It appears that an adequate reserve of stability can be attained by increasing the factor of safety against horizontal sliding by 0.5 (or possibly less), i.e., providing an additional horizontal resisting capacity equal to 50 percent of the existing horizontal hydrostatic thrust. This force would amount to about 15 kips per foot. In our opinion, it would not be necessary to anchor the reconstructed or underpinned sections of the wall.

The system would be installed by drilling a 4.5-inch diameter or larger percussion hole into sound bedrock, placing the girth beam, inserting and grouting a suitable bar or stranded tendon, testing the anchor, and then locking off the anchor at a percent of the design load.

The entire installation probably can be installed below present grade or the grades adjusted somewhat to provide the desired cover to hide the system.

If this scheme is of interest, the preliminary anchor design criteria shown on Table 2 may be used for feasibility studies. The anchor loads may be increased or decreased depending on the most feasible combination of anchor load, spacing, and girth beam section. Where there is minimal

penetration of the wall foundation below grade, a satisfactory result may be achieved by extending the beam below [illegible] the structure as shown in Fig. 4. Prior to adopting this scheme, it should be carefully studied by a structural engineer to evaluate the effects on the existing wall and to develop the most economical structural system. The rock cores should also be studied in more detail to verify anchor embedment criteria.

REGISTERED PROFESSIONAL ENGINEER
RUDY J. DIETRICH
No. 15,685
CIVIL
STATE OF CALIFORNIA

SHANNON & WILSON, INC.

By Rudy J. Dietrich
Rudy J. Dietrich

TABLE 1

ESTIMATED COSTS

EXTENSOMETER SYSTEM

EIGHTH AVENUE RESERVOIR

NASHVILLE, TENNESSEE

		Quantity	Unit	Unit Cost	Estimated Cost
1.	Stainless steel tubing, 0.25-inch with 0.5-inch PVC tube	347	LF	$ 2.02	$ 700.94
2.	Invar rod, 0.25-inch with 0.5-inch PVC tubing	104	LF	3.02	314.08
3.	Anchor, hydraulic expanding	18	Each	55.00	990.00
4.	Linear potentiometer, 0.002-inch sensitivity, 2-inch range	18	Each	310.00	5,580.00
5.	Terminal housing, protective cover	18	Each	Included in Item 4.	
6.	Cable, excluding conduit, junction boxes, etc.	10,000	LF	0.25	2,500.00
7.	Control terminal with 20 channel readout scanner and automatic alarm circuit	one	Each	10,000.00	10,000.00
8.	Inspection and check out, including expenses	20	Mandays	300.00	6,000.00
				Total	$26,085.02

[illegible]

TABLE 2

PRELIMINARY ANCHOR DESIGN AND INSTALLATION CRITERIA

EIGHTH AVENUE RESERVOIR

NASHVILLE, TENNESSEE

Basis for Design

Provide a minimum factor of safety of 1.5 against lateral hydrostatic thrust at anchor yield.

Provide 15 kips per foot anchor resistance at anchor yield.

Assume 25-foot anchor spacing.

Required lateral force 375 kips per anchor.

Required anchor force 375 kips/cos 30 degrees = 433 kips.

Anchor Type

Stranded tendon (VSL type or equal) ER 5-16, 13 strands, area 1.989 square inch.

Ultimate strength (f'_s = 270 ksi), 536 kips
Yield strength (0.85 f'_s = 320 ksi), 456 kips
Factor of safety at yield force, 1.53.

Working force (0.6 f'_s = 162 ksi), 322 kips
Factor of safety at working force, 1.36.

Bond Length

Assume ultimate grout/rock bond, 300 psi.

Assume 4.5-inch diameter hole, sound limestone and shaley limestone.

Required bond length to develop yield force of tendon, 8.9 feet.

Provide a minimum bond length, 15 feet.

Provide a minimum anchor length, 35 feet.

Installation

Percussion drilled hole, 4.5-inch diameter, length 35-60 feet, total length 3,000 LF.

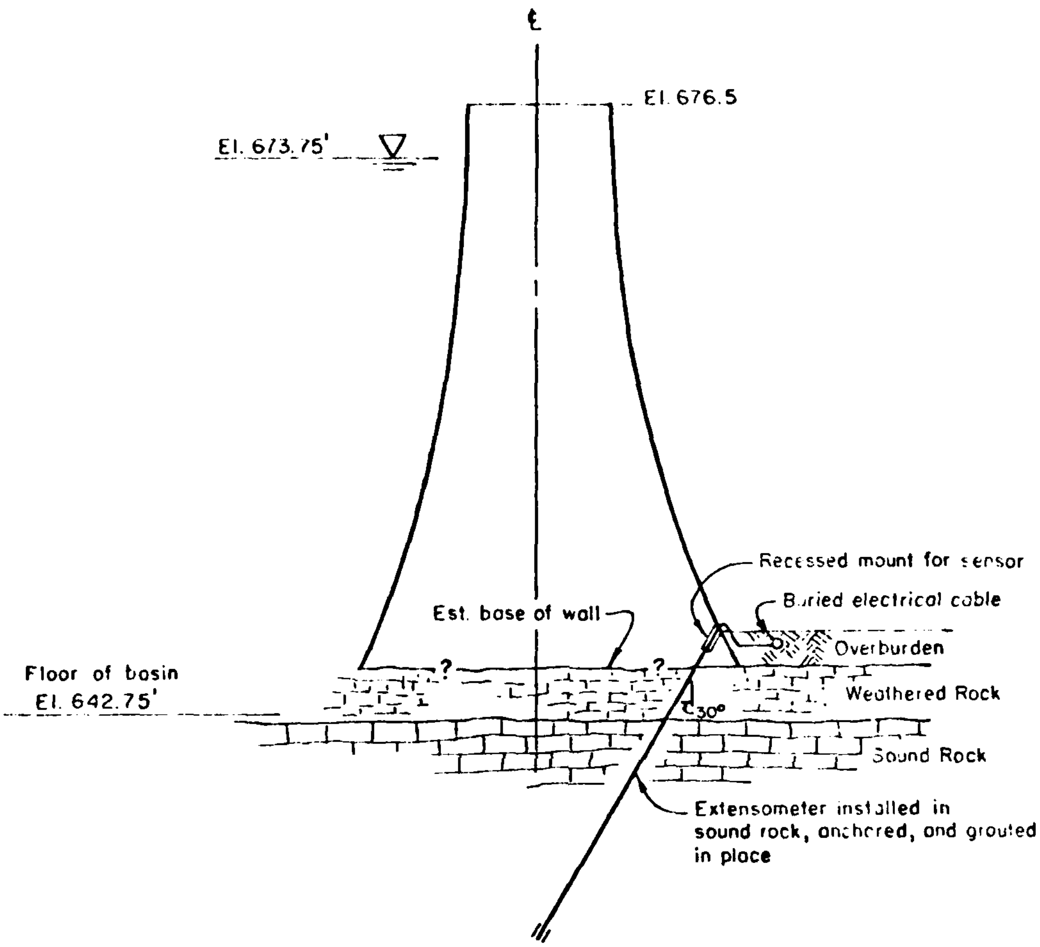
Install tendon and grout anchor.

Place girth beam.

Stress anchor to 322 kips and verify performance.

Lock off anchor at 216 kips, factor of safety at lock off, 1.[illegible].

Grout stressing length of tendon for corrosion protection.



Scale: 1" = 10'

Note: Minimum extensometer penetration 5' into sound rock, or a minimum total length of 20'.

Eighth Avenue Reservoir

EXTENSOMETER INSTALLATION MINIMUM LENGTH

C 395

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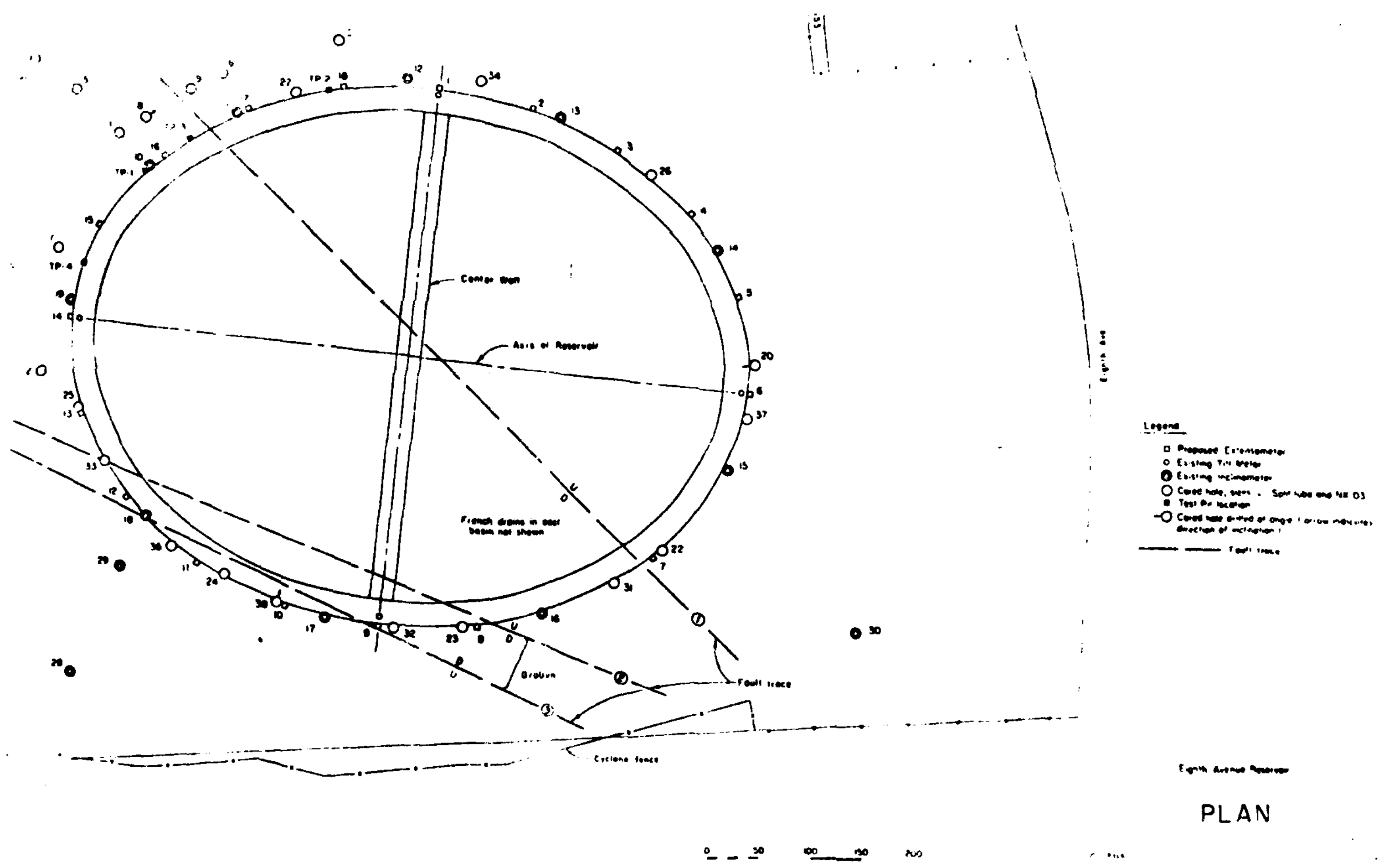
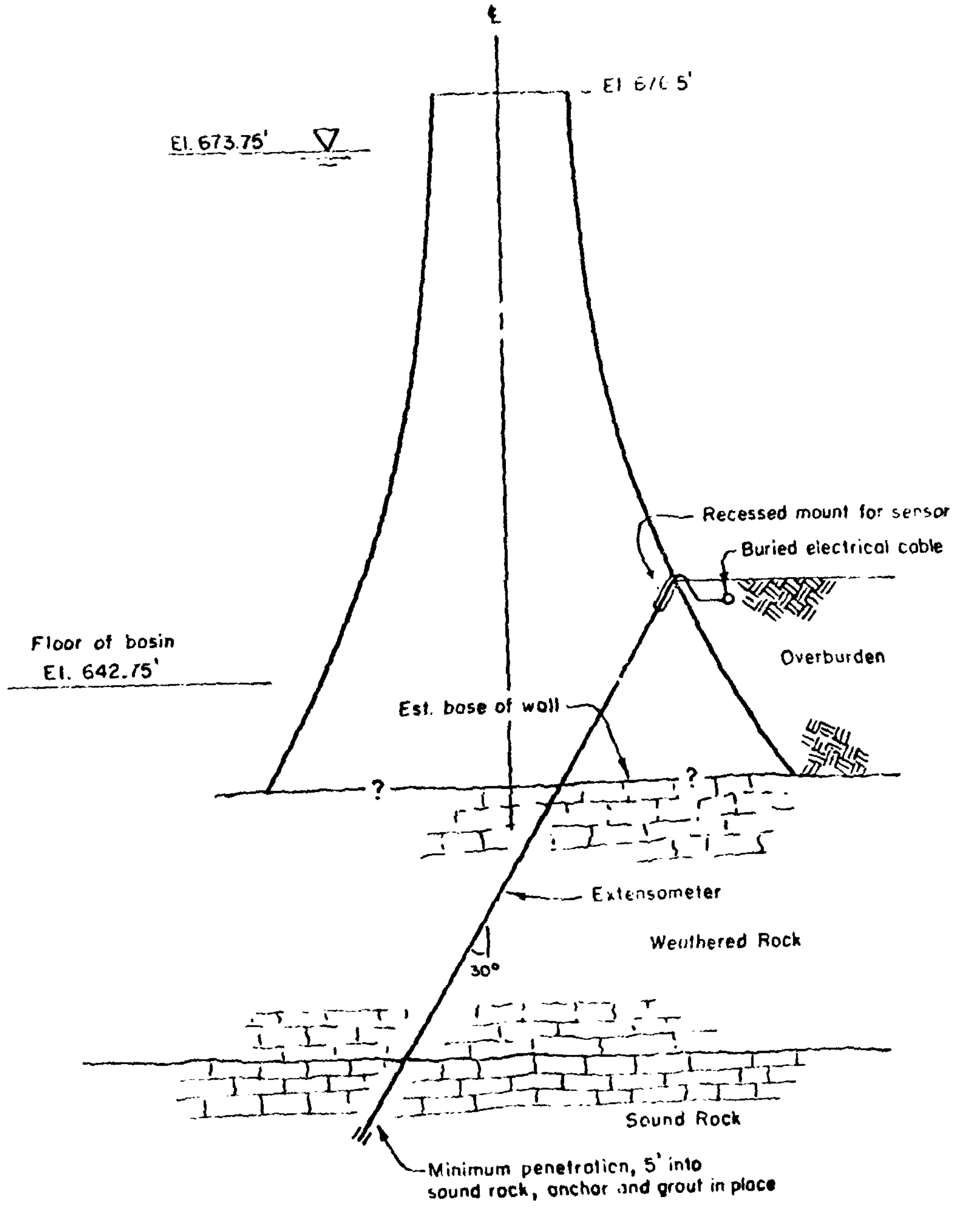


Eighth Avenue Reservoir

PLAN



Scale: 1" = 10'

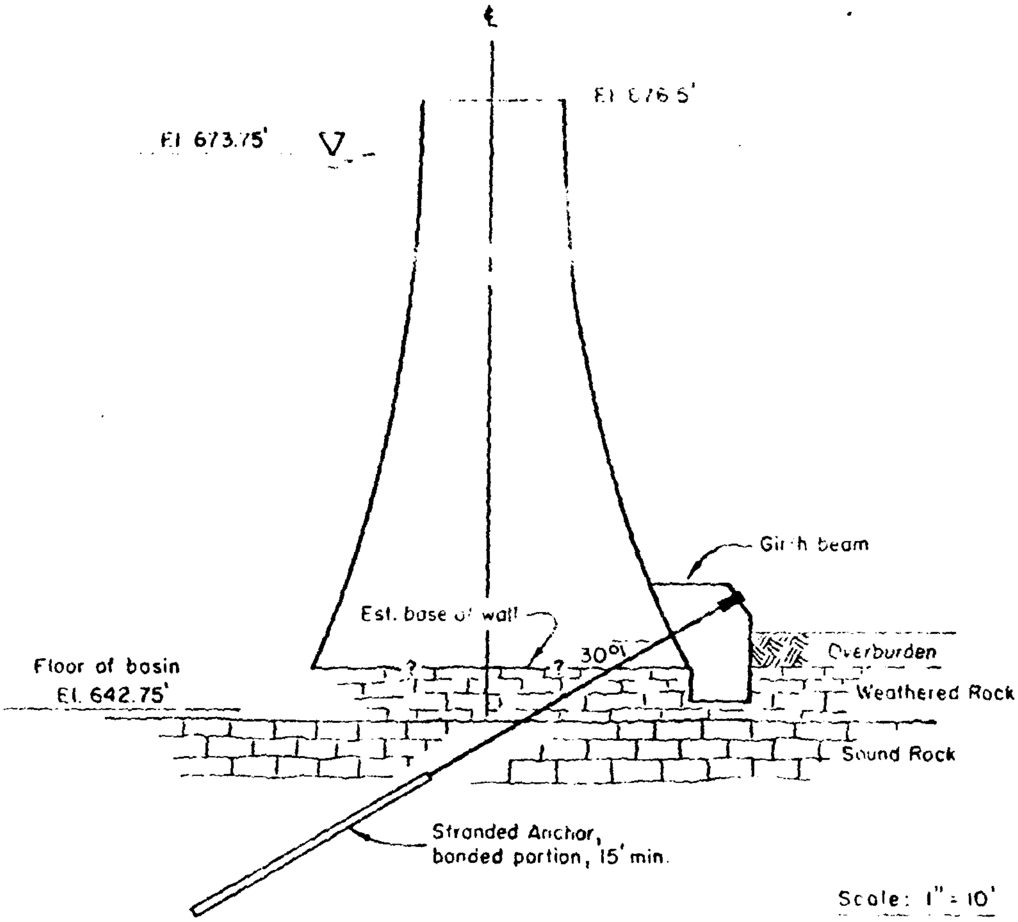
Eighth Avenue Reservoir

EXTENSOMETER INSTALLATION
MAXIMUM LENGTH

C-395 Jan, 1976

SHANNON & WILSON, INC.

GEOTECHNICAL CONSULTANTS



Scale: 1" = 10'

Note: 1. Minimum anchor length, 35'.

2. Configuration of base of wall is unknown.

Eighth Avenue Reservoir

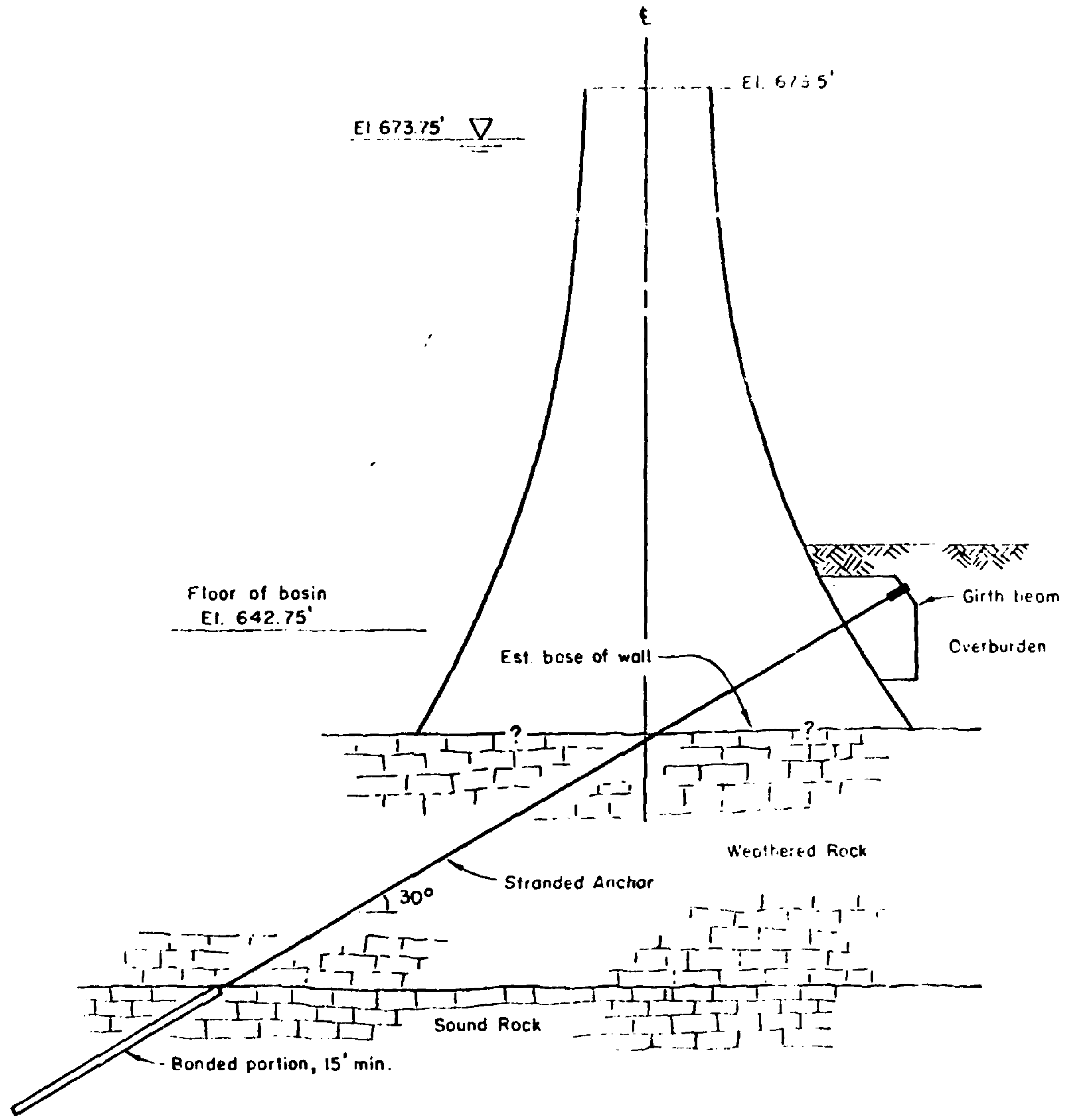
ANCHOR - MINIMUM LENGTH

C-395 Jan, 1976

S[illegible] & WILSON, INC.

GEOTECHNICAL CONSULTANTS

FIG. 4



Scale: 1" = 10'

Note: Configuration of base of wall is unknown.

Eighth Avenue Reservoir

ANCHOR - MAXIMUM LENGTH

C-395 Jan, 1976

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GEOTECHNICAL CONSULTANTS

FIG. 5

APPENDIX C

Letter Report

Eighth Avenue Reservoir

January 19, 1976

William I. Gardner

Geologist

WILLIAM I. GARDNER
GEOLOGIST

7 Archer Circle
Moraga, CA 94556
(415) [illegible]

RECEIVED
JAN 23 1976
GEOLOGIC ASSOCIATES, INC.

January 19, 1976

Mr. R. T. Throckmorton, Jr.
Geologic Associates, Inc.
P. O. Box 668
Franklin, TN 37064

Ref: Eighth Avenue (Kirkpatrick Hill)
Reservoir - Nashville, Tennessee

Dear Mr. Throckmorton:

In accordance with our telephone conversation last December I visited your office and the Eighth Avenue (Kirkpatrick Hill) Reservoir on January 6 and 7, 1976 and I have studied the material you furnished to me by mail and during my visit. Discussions were held on January 6th with you, Mr. L. E. Wilson, of your office, and Mr. Rudy Dietrich, Senior Vicepresident, Burlingame, California office of Shannon and Wilson, Inc.

These general discussions included a review of the reservoir's history, of such information as is available on the wall failure of November5, 1912 and its reconstruction in 1914, the 1920 investigation arising from the discovery of a horizontal crack in the wall just north of the rebuilt segment, the treatment in 1921 by underpinning the wall in parts of the foundation on either side of the rebuilt wall, of matters including possible treatment methods covered in your report of November 25, 1975and an examination of some of the core from selected holes in your recent subsurface investigation. There then followed a discussion led by Mr. Dietrich of possible monitoring systems during which the possibility of using tendons to stabilize the wall and foundation was mentioned. Mr. Dietrich was to give further study to these items.

On January 7th I studied the enlarged photographs of the 1912 failure and additional records from the 1920 and 1921 work. Unfortunately a storm on the 7th prevented any detailed field work but the reservoir was revisited and the beds and faults exposed in vertical cuts along the nearby highway

were viewed briefly. Although these beds are not of the same horizon as those immediately underlying the wall they were viewed as being representative of the geologic conditions at Kirkpatrick Hill.

Since then all of the records have been used for a more detailed study.

DATA ON RESERVOIR

Constructed 1887-1889, masonry wall of limestone blocks and mortar.
Crest elev. 676.5 Width, crest 8feet, at floor of basin 23 feet.
Maximum operating water level 673.5
Base of weir through dividing wall 672.25
Floor of basin 642.75 (top of lining)

Foundation - limestone (Ordovician age), gently dipping northwesterly, contains clay seams in weathered zone parallel with the bedding, contains interbedded shale and 3 small scale faults in reservoir area, beds are crossed by polygonal jointing.

The 1975 investigation was stimulated when seepage apparently increased considerably along lower courses of the wall in its northwestern arc, at the other end from the 1912 failure. There was no seepage at the time of my visit because that half of the reservoir had been drained. However the three lower course of masonry had been darkened by the water as seen in colored photos and there were holes in the mortar through which there had been seepage. The elevation of the top of this zone is about 652.5.

1912 Failure and Treatment in 1920-1921.

The only information on the November 5, 1912 failure are the photographs and there is practically nothing on the reconstruction of the wall in 1914. However the few photographs cover the break fairly well and, although one's imagination must be called upon, a reconstruction of the event may be instructive in considering the present situation.

There were three major segments of the break and, as viewed from left to right on a photo taken from inside the reservoirthey are; Block "A" very roughly 120 feet long, Block "B" about 25 feet long and Block "C" about 70 feet. The break between A and the in-place wall to the north is nearly vertical where-as the contact between the rebuilt and old wall is roughly 1:1. This makes it difficult to locate the segments accurately in their original positions but, as nearly as can be estimated, B rested on fault No. 1 as shown on the Plan, sheet 1 of 6,by

Geologic Associates, Inc, Proj. 75-090, Nov. 24, 1975. Actually the plotted position of the break between A and B is directly over the fault as drawn, with B on the hanging wall side. However the location of the fault too is approximate. In any event the rough location of the fault with respect to these two blocks is important because they are the blocks where foundation failure occurred. At the C segment the lower part of the wall remained intact and was still in place on its foundation as viewed from the reservoir; whether there was some slight shift on its foundation and what was the situation on the outer side cannot be discerned.

The initial failure evidently occurred at B because this largely intact block moved farthest outward, it is leaning sharply inward and the foundation rock appears to have been most deeply affected, either by a mass of it m[illegible] a deep seam or by being deepened by erosion during emptying [illegible] reservoir basin. It is not known whether any part of the foundation remained attached to the base of the masonry but considering the geological conditions it is safe to assume that movement took place on one or possibly more weak seams in the foundation. The moving mass clearly pulled away along joint planes from the rock remaining in the foundation near the perimeter of the basin. Block A pivoted about its northern end, a near vertical, jagged break, and swung outward. It parted and sheared on some of its lower courses on the reservoir side and these remained in place for possibly one third of its length from the north end. Apart from this the rest of the block moved on a seam within the foundation. A close-up photo from outsde the reservoir, of A and the in-place wall shows limestone beds still in contact with the masonry although there is a sharp gorge that was eroded beneath the area of the break. It also appears that the base of the wall was not keyed into rock but was placed at about the same elevation or only slightly lower than the basin floor.

Notes on the exploration work by test pits and drill holesin 1920 disclose northeasterly trending seams in test pits Nos. 2 and 3 in the southeast quarter of the wall. A similar seam in test pit No. 8 near the major axis readily transmitted water as drill holes along the vicinity of the fracture within the reservoir lost drill water which soon appeared in pit No. 8. In the opposite direction towards pits Nos. 2 and 3 this fracture crosses fault No. 1 which probably diverted the water and carried it about 40 feet laterally to escape under the wall as there was no mention of water in the latter pits. The following year features called cutters (joints) and a fault were encountered in the excavations for underpinning the wall. These fractures are similar to and about parallel with the seam found in 1920; they are 40 - 60 feet northwest of the latter.

The fracture system described above provided a geologic situation which could carry seepage from a large area of

the basin through the rock to fault No. 1 and thence under Block B. Over a period of 23 years of operation seepage probably slowly increased and the foundation, particularly at B. was kept saturated by the flowing water and weakened the clay seams. When B failed the dynamic force of the outflow displaced A and simultaneously large blocks of the wall at C failed along masonry courses and were toppled outward. There may have been a sharply increasing seepage flow through the foundation for some time preceding the break but the time from the failure on the clay seam to the failure of the wall would have been a matter of seconds.

When the wall was rebuilt in 1914 an interior perimeter drain was constructed. In 1921, besides underpinning rather short segments of wall, another drain was constructed along the northeast fault, so called, which drained to an outlet at test pit No. 8. There is no record of the flow from these drains or whether they are still functioning.

Present Problem

The latest problem arose when seepage on the wall in the northwest arc increased significantly. It issued through the mortar up to elevation 652.5 or about 5.5 feet above ground level and 10 feet above the reservoir floor. The reason for the seepage to increase after the basin was cleaned and the interior side of the wall gunited in 1974 is obscure. If the seepage was directly through the wall it should have decreased after the wall was gunited. When the reservoir was cleaned there was about one inch of fine material on the floor and removing it could have allowed more seepage loss if cracks in the lining were no longer sealed. If seepage on the wall was due to the artesian pressure of water hydraulically connected to the reservoir and moving through fractures in the foundation rock, then it seems that there would have been some indication of this by the water levels in at least one or more of holes Nos. 1 through 10 which were drilled before the west basin was emptied (June 23-24, 1975).

Most of these 10 holes are 25 feet or more from the wall and some are close to a steep hillside which could allow rapid drainage and pressure relief. Hole No. 10 is at the base of the wall but the evidence at hand is fragmentary and after the basin was drained. No. 10 is on Profile B-B and the rather slow drop in water levels along the Profile and at hole No. 7 after the reservoir was drained suggests a low transmissibility so, unless there was a rapid pressure drop under the wall it seems that there would have been a high piezometric surface at No. 10.

The above discussion was based on the assumption that the base of the wall is flat along a radial line and therefore the lateral flow was unobstructed. As is discussed later this is true for some sections but there are others where the foundation was stepped down toward the outer perimeter of the wall. At such places the wall would act as a cut-off, in this case on the downstream side of the dam, and

cause uplift pressures within the wall.

Further investigation is needed to determine the mode of seepage, ie., if there is movement of water directly through the wall from the the reservoir or if it is artesian pressure under the wall or a combination of both.

Foundation

The core drilling has furnished data from which, in combination with an understanding of local geology, a clear picture of the subsurface geology has been drawn. The core recovery was excellent and the beds can be correlated from hole to hole; thus the displacements accounted for by faults Nos. 1, 2 and 3 are reasonable interpretations. Since the holes were, by necessity, located several feet from the wall the exact elevation of its base is not known and cannot be related precisely to the geologic log. The elevation evidently varies from place to place depending upon rock conditions as is depicted on Profile D-D that follows the perimeter. Due to the flare of the wall however it would have been intercepted by the holes if the base were much lower than has been estimated.

The impression that part of the failed wall was founded on rock close to the elevation of the basin floor is strengthened at a nearby section on a drawing showing the underpinning as constructed (Dwg. 698-31, Aug. 1, 1921, by The Chester Engineers, Pittsburg, Pennsylvania). At some other radial sections this drawing shows the base of the masonry wall to have been stepped down towards the outer perimeter, the vertical differences ranging from 7 feet to 9feet 10 inches. If this was done at other places in the original construction it raises the question of the quality of the rock left in the foundation toward the interior perimeter and also of the possibility of artesian pressure in the wall due to trapped or at least impeded movement of water at any segments where natural drainage may be inadequate.

The limestone is competent rock but clay seams in the weathered zone parallel to the gently dipping beds are serious weaknesses. The 1912 failure is believed to have occurred on such a seam or seams. The foundation generally falls within this weathered zone. The conclusion is inevitable that much of the wall, except for the rebuilt and underpinned segments, is now founded upon rock of dubious quality.

Future Measures

The foundation failure in 1912, the information from the investigations and underpinning in 1920-1921, the present leakage from the wall and the recent subsurface investigation all indicate that there are problems in the original foundation and to an unknown degree possibly in the wall itself. The estimated low factor of safety estimated by Geologic Associates, Inc. is consistent with these elements and indicates the desirability of resolving the problem directly, such as by supporting the wall

on or tieing it to competent rock. Underpinning or reinforcement by tendons are examples of such an approach. They are direct, the construction is controlled and the results are more readily known. In my opinion grouting will not appreciably improve the shear resistance of the clay seams.

Drainage is a secondary measure which by itself would not greatly improve the present stability of the foundation. A drain along the inside perimeter as was constructed along the rebuilt section would offer some protection to the clay seams, it would reduce the hydrostatic head and the flow of water through the foundation at those segments that are not drained naturally. It is probable that along the northern and southern sides which are near steep hillsides the natural drainage is entirely adequate. There is no information on whether the present drains are functioning and this should be determined. Likewise lining the reservoir floor and guniting the wall are highly useful to keep leakage within acceptable quantities and protect the foundation but reducing leakage now will not improve the foundation.

Monitoring the foundation has difficult aspects. The problem is to interpret the data from a monitoring system, to recognize the warning signal and to have an idea of how much lead time it gives as a warning.

Little is known about the condition of the wall itself. After 88 years the mortar is bound to have deteriorated. Drill water at holes drilled from the top of the wall in 1920 was lost and issued along the wall for considerable distances. And of course there is the latest warning signal along the west basin. Any future program of rehabilitation should include testing the wall and determining its condition.

Respectfully submitted,

William I. Gardner

William I. Gardner

Consulting Engineering Geologist

Jw

[illegible]
5 Dunwoody Park, Suite 118
Atlanta, Ga. [illegible]
(404) 394-[illegible]

THE CHESTER ENGINEERS

February 16 1976

RECEIVED
FEB 18 1976
WATER & SEWERAGE SERVICES
Nashville, Tennessee 37201

Mr. K. R. Harrington, Director
Department of Water & Sewerage Services
8th Floor - Stahlman Building
211 Union Street
Nashville, TN 37201

Dear Ray:

8th Avenue S. Reservoir
Foundation Investigations

We are in the process of preparing a report for you on the investigations, discussions, and recommended improvements and modifications on the subject structure. Briefly, that report will cover a brief history, summary of present conditions, analysis of sub-surface investigations, alternative solutions, proposed instrumentation, and recommended program for repair together with preliminary design and estimate of cost.

The investigation and analysis at this date indicates that lining and a system of tendons or tied-back ring beams would be the most feasible. There is, however, one vital item of information presently missing from the investigations performed to date, ie., the structural integrity of the masonry wall itself together with water levels within the walls. To that end, we believe that it would be prudent at this time to core the wall from the top at a minimum of four (4) points. We would certainly want to know the condition of the wall before recommending any substantial anchoring system. It is estimated that such coring work would cost approximately $6,000 to $8,000.

In addition to the remedial work which will be recommended, we will also recommend the installation of an instrumentation system. This system should be installed regardless of any remedial work and we believe the sooner-the-better. Such instrumentation would not interfere with the proposed remedial work and would greatly

2/16/76 - Copy to Bill Brady, Bobby Williams, Gene Johnson
John Upham & Mike Patterson

[illegible]

assist in the final installation and filling of the reservoir.

It has been recommended that the instrumentation consist of a system of rod-type extensometers, anchored into the underlying solid rock at one end and the base of the outside wall at the other. The instrument could detect movement or would have a sensitivity of 0.002 inch with maximum travel of ± 1.0 inch. An estimate of the materials and equipment for the system with permanent read-out and monitoring devices is $26,085. This total does not include installation, i.e., drilling and grouting and engineering cost involved in monitoring the system.

Pending preparation of our report we would be happy to discuss the foregoing items with you to answer any questions you may have. Please let us know if you concur in the foregoing recommendations.

Very truly yours,

THE CHESTER ENGINEERS

T. A. Fithian

T. A. Fithian

TAF/lh

cc: Ray Washburn
Ray Throckmorton

WILLIAM I. GARDNER
GEOLOGIST

[redacted]

7 Archer Circle
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[illegible]

(415) 376 5325

February 5, 1976

RECEIVED
FEB 16 1976
GEOLOGIC ASSOCIATES, INC.

Geologic Associates, Inc.
P.O. Box 668
Franklin, TN 37064

Attention: Mr. R. T. Throckmorton, Jr.

Ref: Eighth Avenue Reservoir
Nashville, Tennessee

Dear Mr. Throckmorton:

Mr. Rudy Dietrich, Senior Vice-President, Shannon and Wilson, Inc., phoned Thursday, January 29th to discuss his report a copy of which I have received. I briefed him on mine of January 19, 1976 and am sending him a copy.

I explained that the main purpose was to present an independent review of and judgement on the geological conditions and the reservoir situation. The dominant question of course is whether the reservoir in its present condition is safe. Unfortunately an absolute answer to that question is unobtainable and the answer must be within the framework of a judgement and an opinion.

I believe that from what we know of the geology and the past history of the reservoir that there is a serious risk in the foundation and possibly to some extent in the wall itself. We cannot measure or state with certainty the degree of risk, the odds might be that the reservoir would stand there another quarter or half century with only minor problems, on the other hand a section might fail over-night as it did in 1912, after 23 years of operation. I think it fair to point out also that Baldwin Hills Reservoir, in Los Angeles, had a foundation failure after many years of trouble-free operation and that it provides a case history of the astronomical cost of such a failure within a city.

On the question of whether a monitoring system would enable early use of the West Basin prior to any reinforcement measures I do not believe that an opinion should be given without much more detailed information on the foundation and the wall and on water table and piezometric levels in both. Then if the situation suggested that a monitoring system might be a sufficient temporary safeguard, it would be necessary to test the system over a trial period to obtain a record of changes, if any, that occur at the West Basin while empty, and at the East Basin under operating

conditions. This prior record would provide guidance in establishing a standard of acceptable phenomena and of criteria for determining the danger signals. The monitoring system as I now envisage it would include the extensometers, water table and piezometer levels from recorders at key observation holes and the tiltmeters, along with a reservoir hydrograph for each basin. I think that before putting your trust in such a system you would need to have more information from core drilling the wall and underlying foundation, from test pits and possibly some tunnels under the wall to study directly the clay seams in the poorest sections. To me it looks that this will add up to considerable cost without assurance that it will work. Furthermore it does not solve the problem.

Thus it appears to me that the ~~mos~~ direct resolution of the problem, assuming favorable estimates of cost, by anchoring the wall to the foundation or otherwise improving the factor of safety will be the approach to take. It is probably the one having the lowest ultimate cost and it avoids taking risks that are unmeasurable but could be very serious.

Sincerely,

William I. Gardner

William I. Gardner
Consulting Engineering Geologist

copy to Mr. Rudy Dietrich
1550 Rollins Road Suite F
Burlingame, CA 94010

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FEB 16 1978

GEOLOGIC ASSOCIATES, INC.

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FEB 16 1976

WILLIAM I. GARDNER
GEOLOGIST

[redacted]

GEOLOGIC ASSOCIATES, INC.
7 Archer Circle
Moraga, CA 94556
(415) 376 5325

February 6, 1976

Mr. R.T. Throckmorton Jr.
Geologic Associates, Inc.
P.O. Box 668
Franklin, TN 37064

Ref: Eighth Avenue Reservoir
Nashville Tennessee

Dear Ray:

Enclosed is my statement for professional services and expenses. Actually I put in more time than shown but there was considerable wheel spinning after I got home because of being under the weather, as they say out here, so I have discounted part of it.

My enclosed letter may be helpful to you to reinforce my report which I kept very brief, but the letter expresses more of my concern for the risks and consequences at the reservoir. I am concerned too for the responsibility of officials in a public agency to lean over backwards in not exposing the public to unnecessary risks. I am in favor of nuclear power plants despite the public uproar because I believe that there is a redundancy of built-in safeguards but in some other less publicised situations it is easy for economics, time, costs, etc. to cloud the main issue.

My skepticism about relying on a monitoring system is due mainly to the uncertainties involved - how much time will you have? Enough to evacuate the basin? Should you also evacuate the homes? And when I compare the relative costs of treatment and the consequences of failure at the reservoir the balance, to me, is overwhelmingly in favor of treatment. At Baldwin Hills the failure by erosiontook a fairly long period of time before the flood was released; plenty of time for a complete evacuation of the people except for one or two, as I recall who absolutely refused to leave their homes and were drowned. The property costs were enormous. I saw another dam, Lower San Fernando, that failed but did not lose the reservoir and cause a great tragedy only because the reservoir was at a low elevation and the quake did not continue for only a few seconds longer. I saw Malpasset three or four months after it failed and the devastation was unforgettable. I have not

seen Vaiont but I have given considerable study to the slide in connection with another consulting job. Those in charge were playing around with it, to ease it down gently you might say and to [illegible] it out as the reservoir was raised and lowered. No one ever thought that it would let go instantaneously and virtually displace the reservoir. Engineers were sentenced to jail because of Vaiont.

These other reservoirs I have mentioned were much larger than Nashville which relatively is only a teaspoonful in volume but any loss of life could have unpleasant consequences for those in charge. I don't want to be an alarmist but people should be aware of these cases if they are not already familiar with them.

The difficulty in monitoring climaxes when the responsible individual must decide - Do I order immediate evacuation or wait? You probably had a somewhat traumatic experience last year in this respect. It is the dilemma now facing the USGS's earthquake people. Last year their monitoring system said "Earthquake coming", so what to do? Send out an immediate warning and scare the people half to death, clog the roads with refugees or wait for further confirmation and hope that things turn out well? In this instance the GS cogitated and finally started the warning message through channels. When the "OK to issue" was received at Menlo Park, CA, a 3.5 quake had just occurred, the warning was not issued, the shock was scarcely noticed and the GS was off the hook.

There is a minor correction in the Table, p. 2 of my report: the max. operating water level should be elevation 673.75. And on the same page, middle paragraph, is the elevation 652.5 correct for the top of the leaky zone?

I hope that things are going well for you and family. We finally got some winter weather and most emphatically because the moisture came down as snow and that is not supposed to happen around here.

With my best regards,

Sincerely

Bill

William I. Gardner

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FEB 16 1979

[illegible] ASSOCIATES, INC.

GEOLOGIC ASSOCIATES, INC.

GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

February 20, 1976

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee 37201

Attention: Mr. K. R. Harrington, Director

Gentlemen:

Re: Supplemental Report
Eighth Avenue Report
Project No. 75-090

There follows a resume' of conferences, the acquisition of further data, our comments and conclusions, and miscellaneous other aspects related to this project which have occurred since our report of November 25, 1975.

EXPLORATION AND TESTING.

We have performed further subsurface exploration which included drilling and pressure testing of four holes (40, 41, 42, 43). These holes were drilled along the perimeter of the reservoir at locations which would cover a representative range of subsurface conditions. The newly acquired data has been added to the geologic profiles which have been reprinted and are submitted herewith. Two physical test data sheets are also enclosed; they should be added to Volume II, Appendix A, of our report.

The core from Hole 41, drilled within the reach repaired in 1914, as expected, exhibited only the most modest of weaknesses, and leaked virtually not at all, especially below the top of bedrock. We point out that the intent of the hydraulic pressure testing was to indicate leakage in a qualitative, rather than quantitative, fashion. We did not wish to actually determine the maximum amount of water which could be made to pass through the bedrock weaknesses at a given pressure.

Hole 43 was drilled almost opposite 41 across the basin, in a reach of the foundation which is only moderately weathered. Again, the lower portion of the hole where weaknesses were confined to softened shale bands and laminae, was tight. The poor condition of the upper portion of the bedrock and the fact that casing had been reamed to about elevation 637, caused some physical problems with the packers which precluded a good test on the upper portion. However, the zone above 637 would have assuredly leaked.

Holes 40 and 42 are outside of the east and west basins, respectively, and essentially opposite each other. Both holes encountered highly degraded bedrock. As was to be expected, the unweathered bedrock did not leak under pressure. On the other hand, above about elevation 615,

the bedrock accepted water freely under an induced head. The consumptions and pressures noted represent the test pump capacity at a preset throttle and RPM. We did not attempt to raise the test pressures (and consumption) to the permissible limits by accelerating the engine! We were surprised to find that during testing of the interval = 615-620, Hole 42, the ground was actually "sucking" water faster than the pump could supply it! This fact seems to indicate that the bedrock openings, although individually small, are quite well interconnected. These aspects are considered further in this report.

ADDITIONAL CONSULTANTS.

After our meeting of December 17, 1975, we requested permission, subsequently granted, to obtain additional opinions regarding the condition and stability of the reservoir, the need for further data, and to provide supplemental expertise in studying monitoring and remedial systems. Subsequently, we arranged for Mr. Rudy J. Dietrich of Shannon and Wilson, Inc., and Dr. William I. Gardner, Consultant, to visit the site and confer with us during the period January 5-7, 1976. We talked with these two persons because of their diverse, but highly

experienced, backgrounds with reference to the geological, engineering, stability, monitoring and long-term repair methods for similar water-retention structures. Messers Gardner's and Dietrich's reports and personal data have been submitted as they became available. We shall comment as necessary on various aspects of their reports in our discussion which follows:

PRESENT CONDITIONS.

The reservoir remains in operation with the East Basin in service and the West Basin essentially drained. Continuing monitoring of the slope indicators and tiltplates does not indicate detectable instability. Recent water level readings in the ten holes still available for observation agree very closely with those recorded since the West Basin was taken from service. Of course, with the West Basin drained there is no seepage through the masonry on that arc; however, minor seepage continues through lower courses between the 1914 reconstruction and the south minor axis. From time to time we have visually inspected the manholes providing access to the 1914 East Basin perimeter drain and the 1921 drain. While water stands in both, it appears stagnant and we doubt that it is possible to positively determine whether either drain system is in fact, functioning. We believe that the recent

pressure test data and the static water level observations tend to confirm our supposition that the major portion of the continuing volume of leakage from the reservoir can follow steeply inclined paths downward and away from the base of the structure through openings in the highly degraded zone. On the other hand, the phreatic level is undoubtedly quite high and slopes gently away from the perimeter of the structure where the transmissibility of the bedrock is poor. We also conclude that only in the latter areas is it likely that any hydrostatic uplift can be exerted at the base of the wall.

Messrs Gardner and Dietrich have concurred in our assessment of geological conditions and their probable effect on past, and future, failures of the structure. They also agree as to the probable mechanisms acting during past failures, as well as those which could influence future failures. Both Dietrich and Gardner take even more conservative approaches to assessing the present stability of the structure than we had taken. Stability calculations which consider hydrostatic uplift forces further reduce the theoretical margin of safety under which the structure is operating. Also, by virtue of both consultants' extensive experience

In considering seismic risks to structures, they have emphasized the fact that even a very modest earthquake could initiate changes in the existing regime which could rapidly lead to failure.

With futher reference to the present integrity of the structure, both consultants agree that some factual data regarding the strength of the masonry should be obtained. We concur, and you will recall that we have deferred this operation for various reasons since last summer. We still have mixed feelings regarding the need for data versus the risks to the structural integrity. Coring the wall will require obtaining nominal 6" ϕ core (≈ 8" ϕ hole) of the masonry and a short segment of the underlying bedrock. Drill water will, of necessity, be introduced into the masonry wall and into the weathered bedrock directly below. It will probably be prudent to have a grout plant standing by during this phase of the exploration. It will be necessary, ultimately, to grout-up these test holes, anyway.

<u>FURTHER CONSIDERATIONS, REMEDIAL MEASURES.</u>

Both Gardner and Dietrich have considerable misgivings regarding the efficacy and safety of a pressure grouting program as a permanent remedial measure. The concensus seems to be that a consolidation

grouting program would be much preferred to curtain grouting of the toe area of the wall (dam). Consolidation grouting would require removal of the butyl cover and access to the floor of each of the basins. All agree that any grout program would require inclusion of a drainage system at its exterior (at or under the toe of the wall) in order to preclude the build-up of hydrostatic forces under the wall, regardless of whether or not the wall foundation is stepped. The drainage system could well be very costly and difficult to install. They further agree that it may be essentially impossible to assess the degree to which the grouting process would enhance the stability of the weathered rock zone.

In addition to our discussion early in January with Dietrich and Gardner, representatives of Metro Water and Sewerage Services, the Chester Engineers and Mr. Roger Bethel of Spencer, White and Prentice, Inc., conferred at our office January 27, 1976. At that time we further discussed the relative merits of the possible remedial efforts mentioned in our report, as additionally defined during our conference December 18, 1975. It was the concensus that both conventional underpinning and a drilled pier

retention system are uneconomical solutions. Further, because of excavation costs and the fact that the reservoir cannot be removed from service for the forseeable future, a general enlargement and reconstruction of the reservoir is not feasible, either.

Everyone seems to agree on the need for one aspect of the remedial work - waterproof lining of the basins. The informal discussions have considered remodeling the existing covers to serve as liners. This procedure seems to be a prerequisite for any of the repair schemes and should probably be implemented as soon as possible. Obviously, converting the covers to liners requires installing a new cover, ultimately. We strongly suggest that the lining be initiated soon and that it not be contingent on ordering and installing a new cover.

During the various meetings several persons have suggested investigating the efficiency and cost of a tie-back retention system. This scheme appears to have several advantages over others in that cost ought to be within reason, it can be installed without interrupting use of the reservoir, and the system can be designed to enhance the operating safety factor to a level acceptable by today's standards. Design and cost aspects of this system are under consideration by Chester.

INSTRUMENTATION.

Prior to returning the West Basin to service, and certainly before beginning any remedial construction, the concensus is that the installation of further instrumentation is essential.

The observation wells and inclinometers installed during the exploration are of continuing use, but need to be supplemented with a system of inclined extensometers which will be capable of detecting very small increments of movement. Mr. Dietrich has discussed this aspect rather thoroughly in his report; in fact, we engaged the services of Shannon and Wilson primarily because of their expertise in designing sophisticated monitoring systems. We believe that their approach is the best available and that the system will function as described. The extensometer system will not only monitor the reservoir subfoundation, but will also be of use in safely installing a permanent retention system, such as the tie-backs or tendons.

Dr. Gardner's concern and comments regarding the "difficult aspects" of properly monitoring the subgrade of the reservoir and in assessing the resultant data, are valid and well-taken. On the other hand, Mr. Dietrich believes that the detection of the small movements preceding any large-scale

movements is essential in providing, "an advance indication of imminent failure." Obviously, the uncertainties pointed out by Dr. Gardner regarding when and how to "sound the alarm" based on the data, as well as the fact that the alarm and subsequent failure might occur within seconds, pose very serious problems which cannot now, or may not ever, be resolved.

While we all have reservations regarding the ability of any monitoring system to provide us with 100% fool-proof or fail-safe data, we believe we must have the best available if we are not possibly to later be thought derelict in our duty to the public safety.

COST ESTIMATES.

Mr. Fithian's letter of February 16th notes the costs we have estimated for the core drilling of the wall.

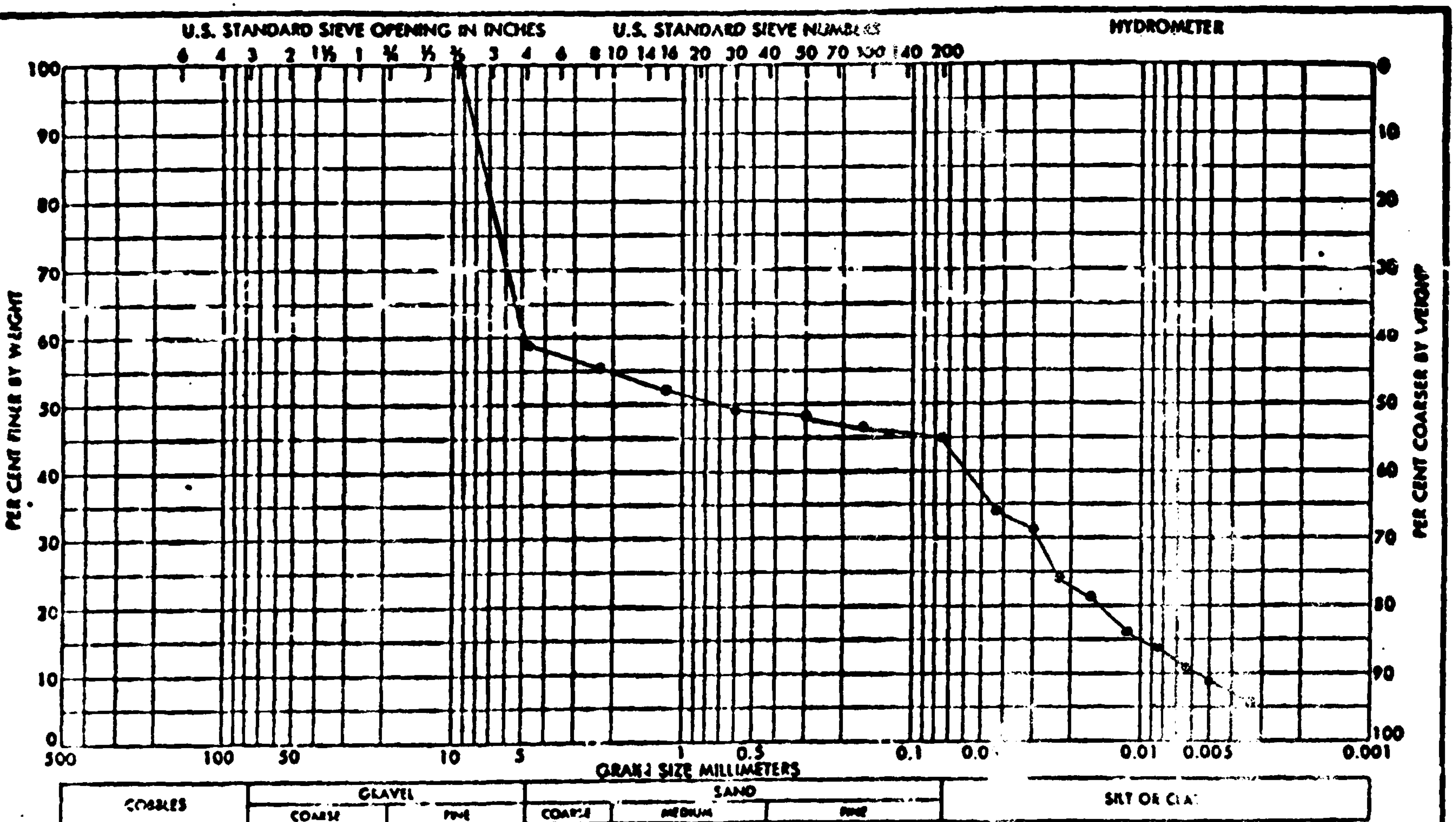
Within a few days we expect to have an estimate of the cost of drilling and grouting-in the extensometers, which will then complete the cost estimate for the monitoring system. We are available to discuss any of the aspects of this matter at your convenience.

Respectfully,

GEOLOGIC ASSOCIATES, INC.

R. T. Throckmorton, Jr., P.E.

RTT/mr
Enclosures: Drawings, 6
Data, 2



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
1	630.2-630.0	Clay, Silty, Sandy, Light Brown; Few Fine Rock Fragments	13.7	25	17	8

GRADATION CURVES

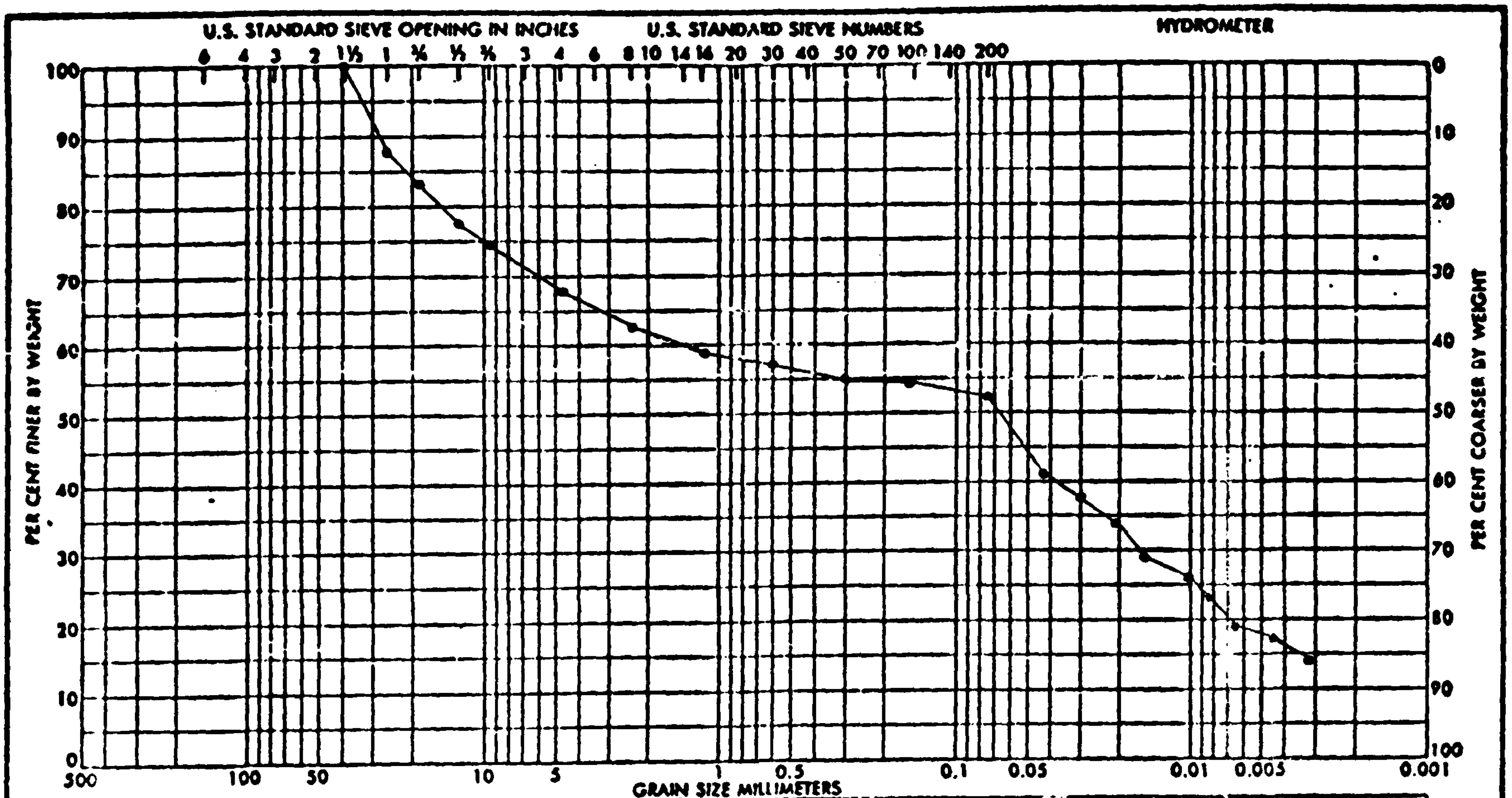
PROJECT 8th. Avenue Reservoir
Nashville, Tennessee

AREA

BORING NO. 40

DATE January 13, 1976

By: Geologic Associates, Inc.
Franklin, Tennessee
Project No. 7-[illegible]



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
2	628.5-628.0	Clay, Silty, Sandy, Light-Brown; with Few Fine Rock Fragments	14.4	29	19	10

PROJECT 8th. Avenue Reservoir
Nashville, Tennessee

AREA

BORING NO. 40

DATE January 13, 1976

GRADATION CURVES

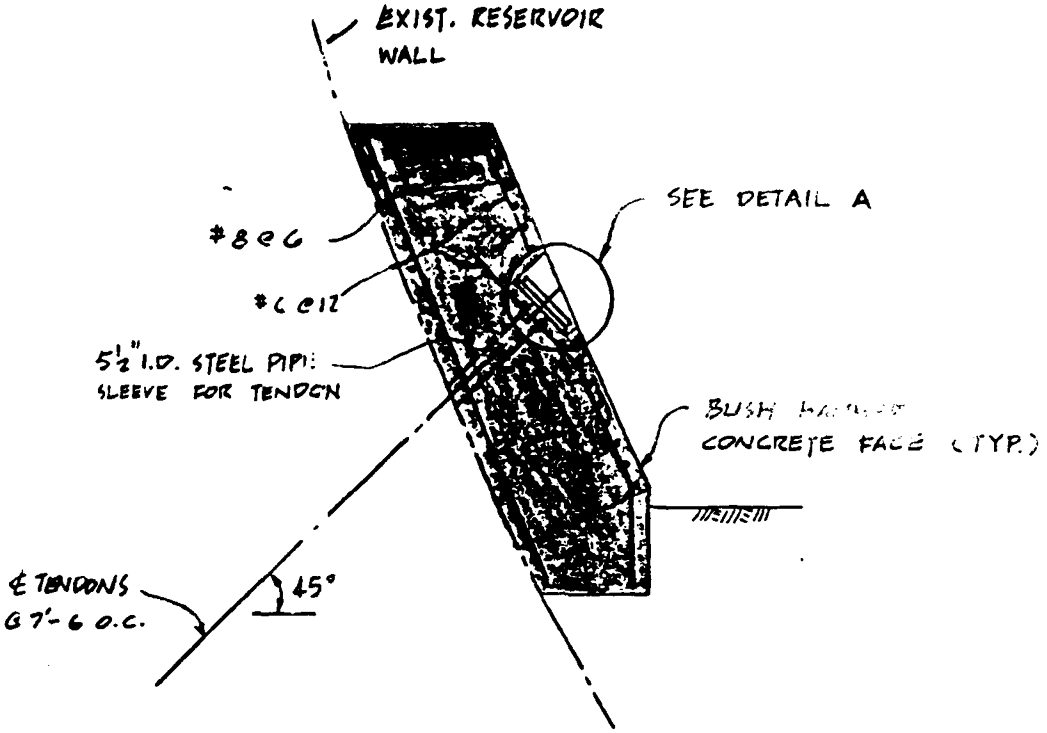
By: Geologic Associates, Inc.
Franklin, Tennessee
Project No. 7?-090

E CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

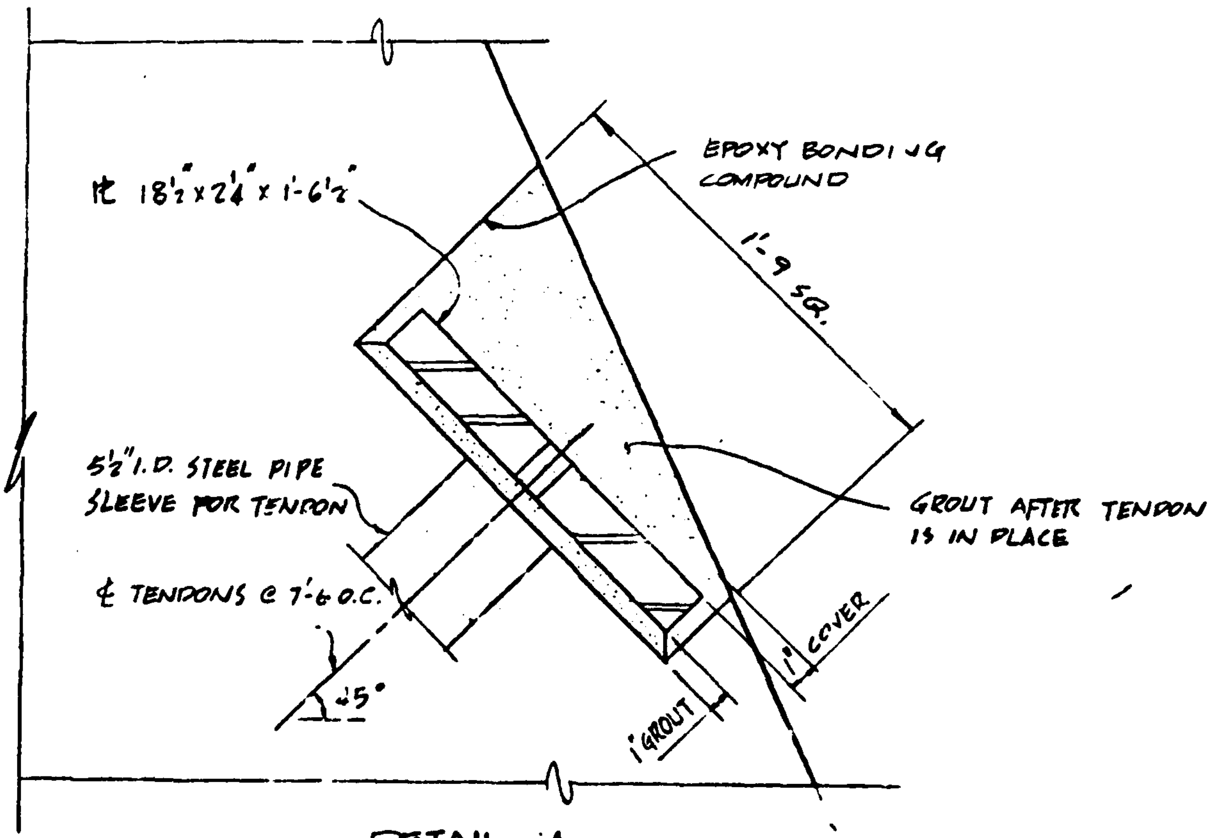
SUBJECT

SHEET NO. 2

COMPUTED BY TCC CHECKED BY E.W. DATE 2/9/77



TYPICAL SECTION AT TENDON



DETAIL A

Prestressed tendons @ 15'-0 O.C.

Check beam reinf for 45° anchoring :

Design as beam spanning 15' :

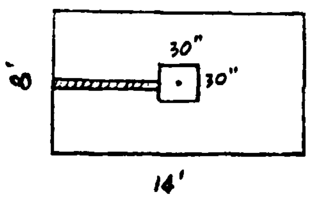
$w = 15/0.707 = 21.2$

$M = \frac{1}{9} \times 21.2 \times 15^2 = 530^{K-1}$

$As = \frac{530}{1.44 \times 32.5} = 11.3^{\square''}$ (14-#8)

Design as spread footing (8'×14') for testing load 477^K :

pressure between wall & Beam : $477/_{8\times14} = 4.26^{KSF}$ //



check shear :

$4.26 \times 8 \times 3 = 102.3^{K}$

$v = 102.3/8\times12\times32.5 = 32.8^{PSI}$ (O.K.)

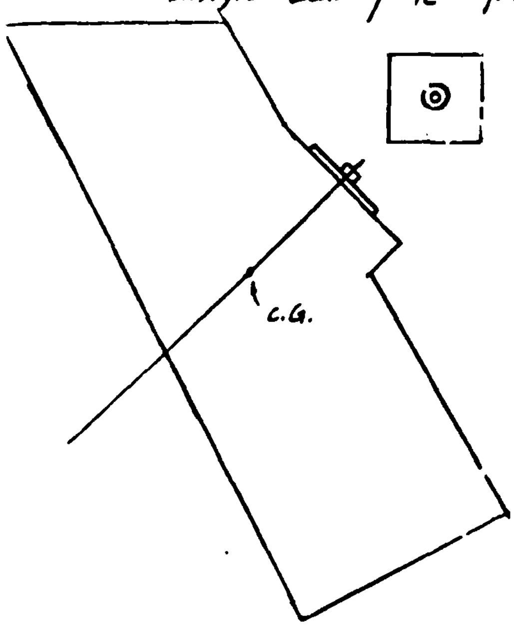
$M_1 = \frac{1}{2}\times4.26\times5.75^2 = 70.4$

$As = 1.5^{\square''}$ Use #8 @ 6

$M_2 = \frac{1}{2}\times 4.26\times 2.75^2 = 16.1$

$As = 0.35^{\square''}$

Design bearing PL for tendon .



$P = 477^{K}$

$A = 477/0.75 = 636^{\square''}$

Use 26×26

$f_p = 477/26^2 = 0.706$ ksi

$M = \frac{1}{2}\times 0.706 \times 13^2 = 59.7^{K-''}$

$27 = \frac{59.7}{\frac{1\times t^2}{6}}$ $t = 3.64''$ (N.G.)

Use stiffeners .

Try

Prestressed tendons @ 7'-6 o.c.

Design as beam spanning 7'-6

$W = 15/0.707 = 21.2\ ^{K}/_{1}$

$M = \frac{1}{8} \times 21.2 \times 7.5^2 = 1$[illegible]

$As = \frac{132.5}{1.44 \times 32.5} = 2.83\ ^{\square''}$ Use #[illegible] @ 12

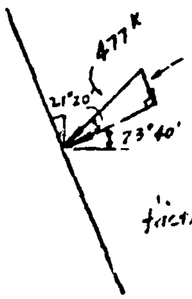
Design as spread footing (8'x 7'-6) for testing load 239^K

$23[illegible]/8 \times 7.5 = 4.0$ KSF

$M = \frac{1}{2} \times 4 \times 3.08^2 = 19.0$

$As = .41\ ^{\square''}/_{1}$ Use #6 @ 12

Find friction force between wall and beam :



spred footing 8'x 14'

$P_f = 477 \sin(21°20')$

$= 477 \times 0.36379$

$= 173.5^K$

friction $f = 173.5/10 \times 14 = 1.24$ KSF

+ Weight beam $45 \times 15 \times \cos 23°40' / 10 \times 15 = 0.41$ KSF } 1.65 KSF

Beam :

$P_f = 21.2 \sin(21°20')$

$= 7.7^K$

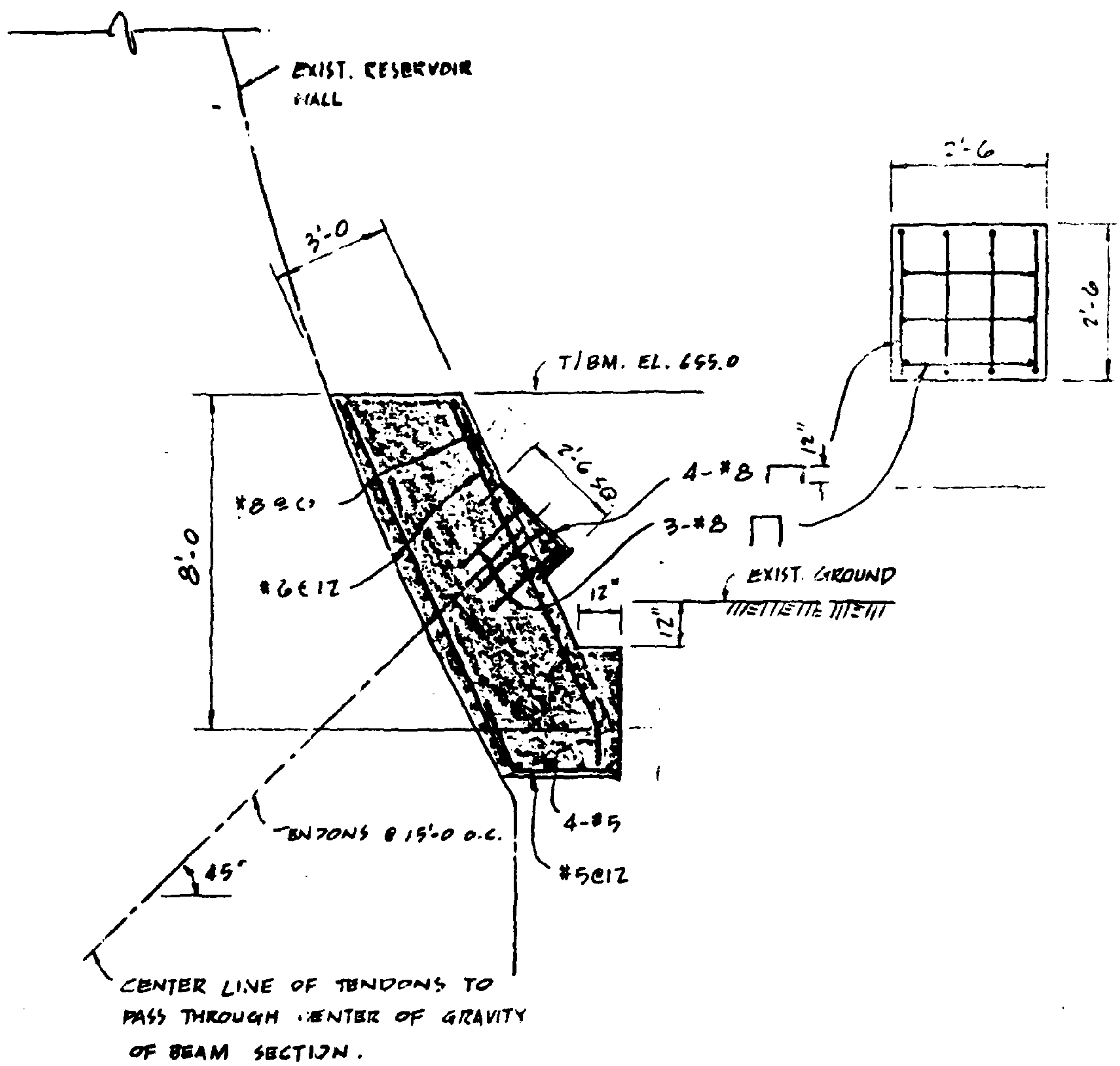
friction $f = 7.7/8 = 0.96$ KSF

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

	TENDONS @ 15'-0 O.C.	TENDONS @ 7'-6 O.C.
MAXIMUM TESTING LOAD IN TENDON	477^K	239^K
ACTUAL PRESTRESS LOAD	318^K	159^K
PRESSURE BETWEEN WALL & RING BEAM	4.3 KSF	4.0 KSF
FRICTION BETWEEN WALL & RING BEAM	1.7 KSF	1.6 KSF

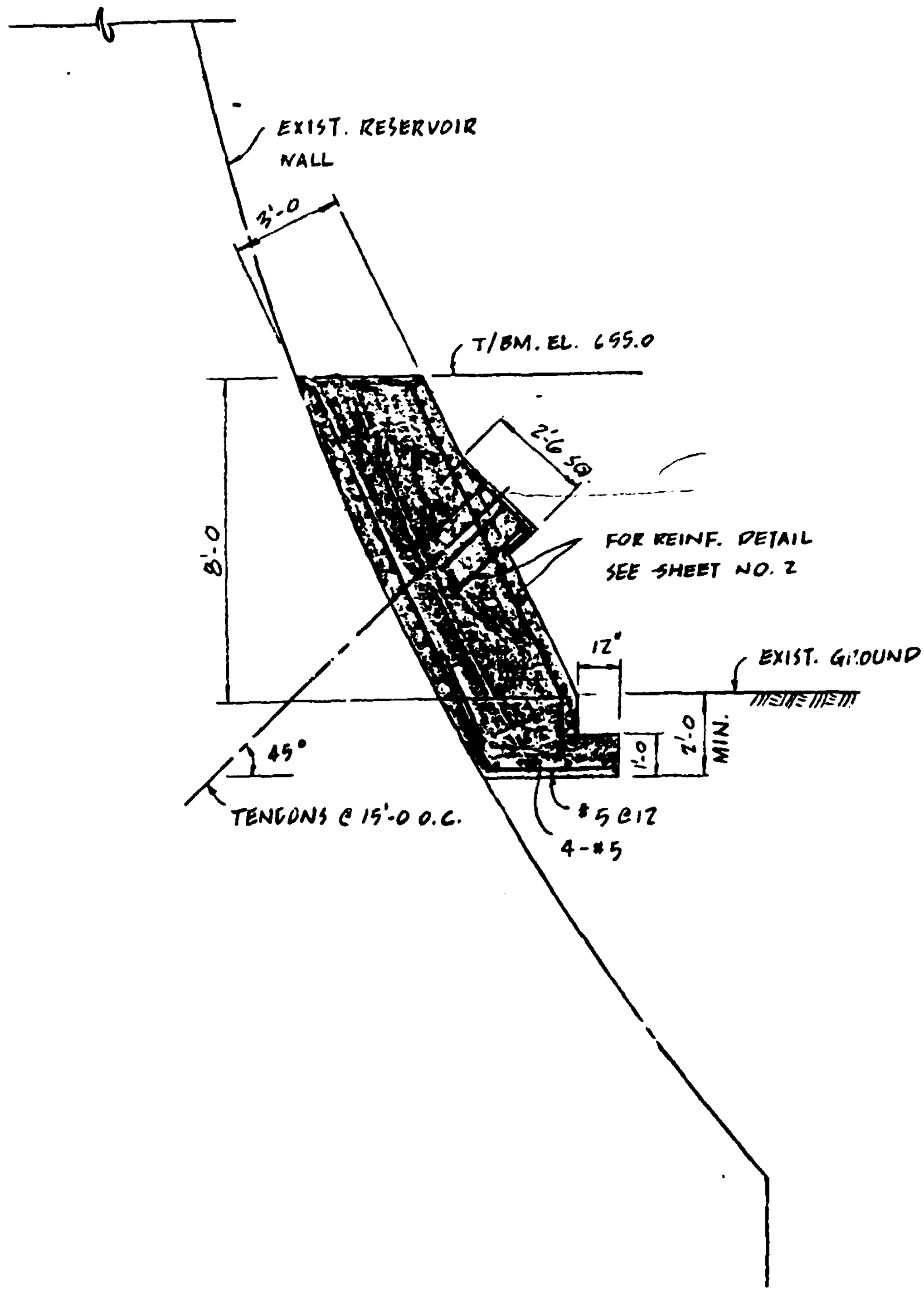
THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT Nashville (8th Ave Reservoir) FILE NO.
SHEET NO. 2 OF 5
COMPUTED BY TCC CHECKED BY DATE 12/14/76



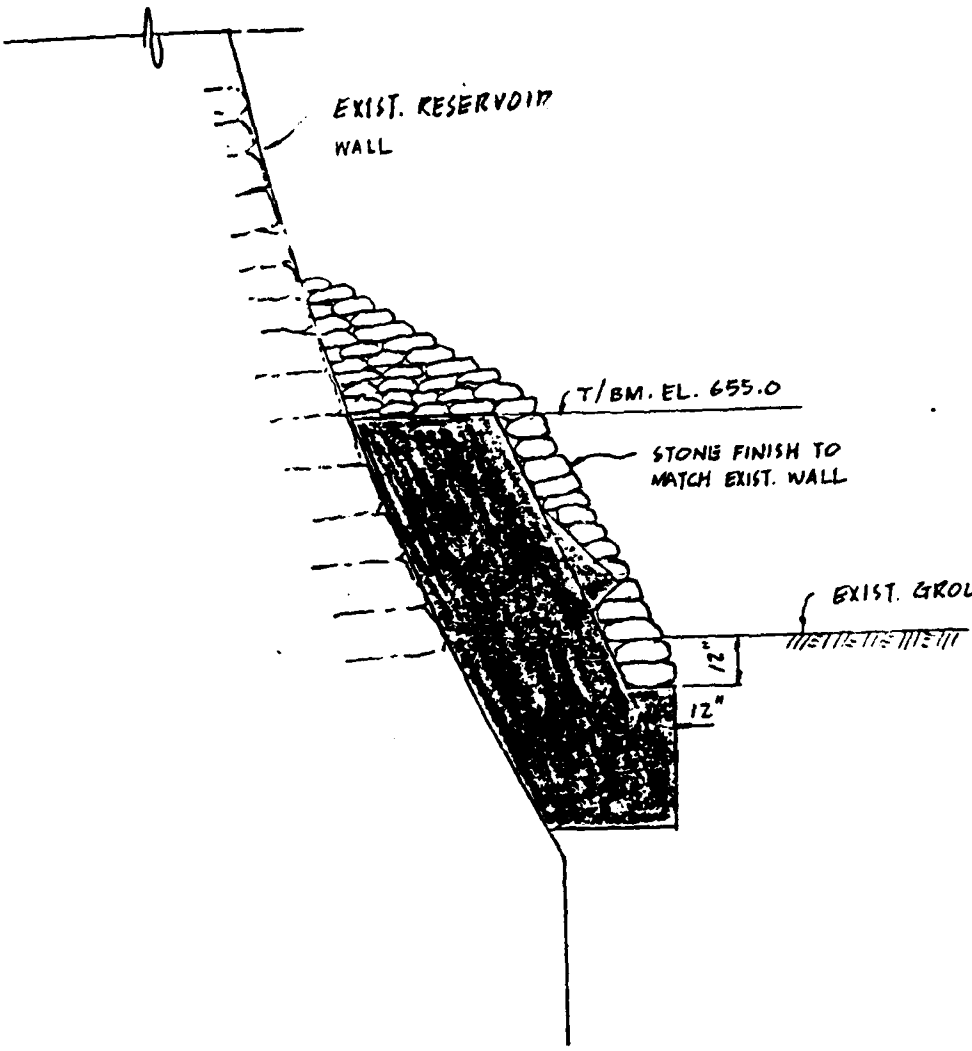
THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT Nashville (8th Ave Reservoir) FILE NO
SHEET NO 3 OF 5
COMPUTED BY TCC CHECKED BY DATE 12/14/76



THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT: Nashville (6" Ad [illegible])
FILE NO:
SHEET NO: 4 OF 5
COMPUTED BY: TCC
CHECKED BY:
DATE: 12/14/76

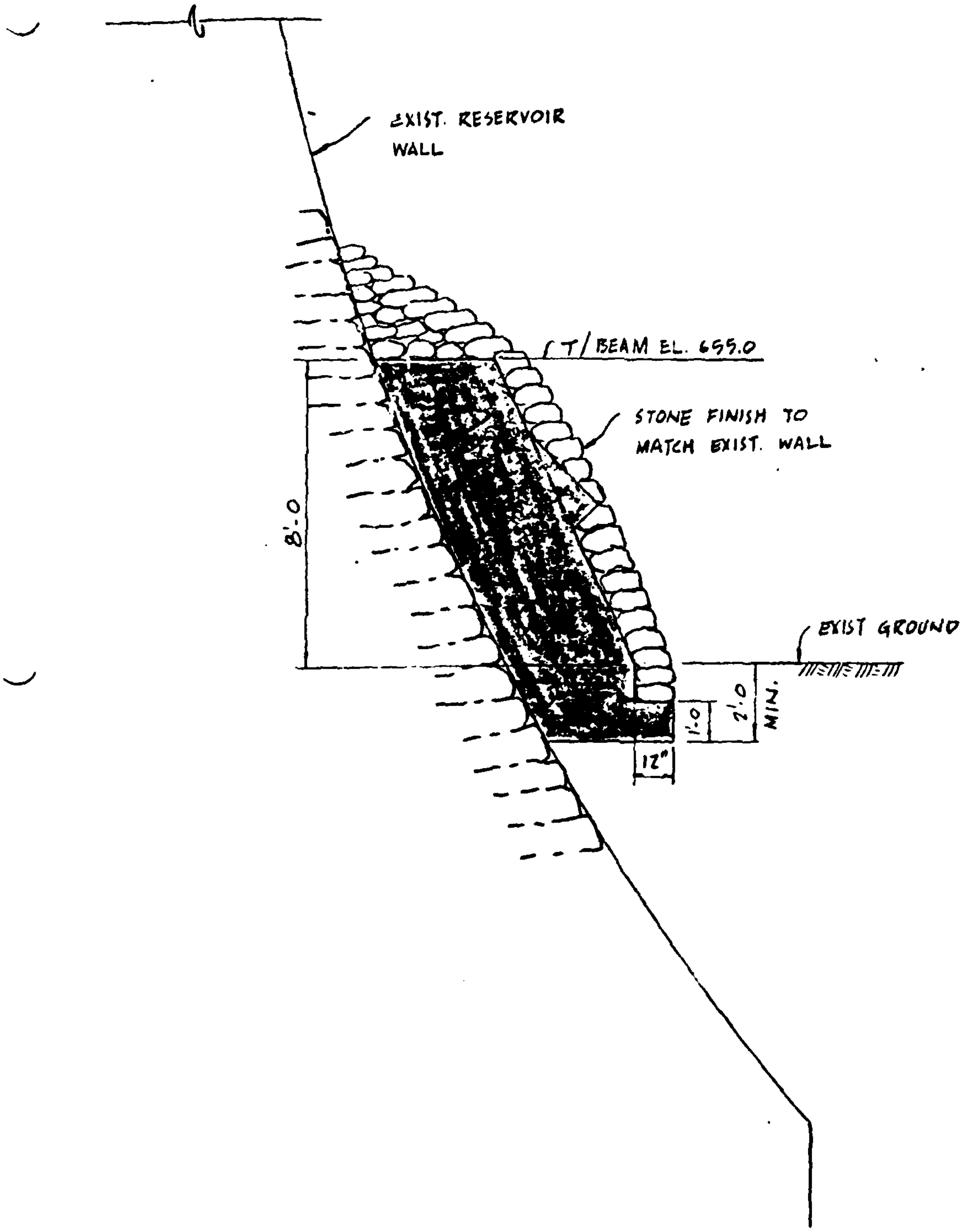


THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT Nashville (8th Ave. Reservoir) FILE NO

SHEET NO 5 OF 5

COMPUTED BY TCC CHECKED BY DATE 12/14/76



THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

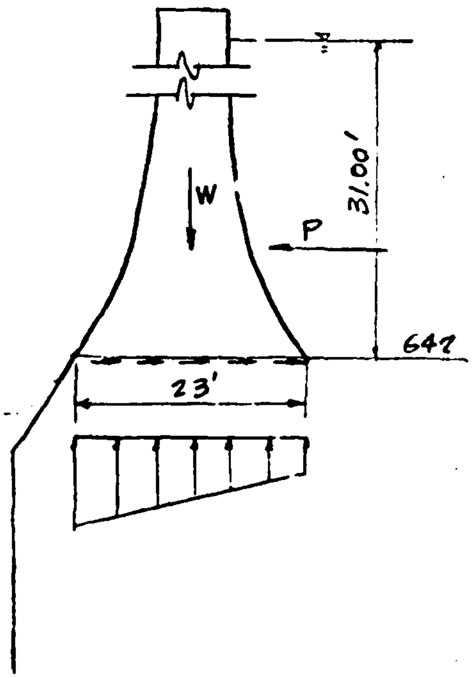
SUBJECT Nashville
Kirkpatrick Hill Reservoir
COMPUTED BY .... CHECKED BY ....
FILE NO
SHEET NO 3
DATE 12/5/75
2/2/76

TOP OF WALL : 676.50
BASIN FLOOR : 642.75
WATER ELEV . 673.75

Water pressure / ft .

$P = \frac{1}{2} \times 62.5 \times 31.00^2$

$= 30.0^K$



HE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT (PRELIMINARY) SHEET NO 4

COMPUTED BY .... CHECKED BY .... DATE 2/4/76

safety factor for existing structure = 1.2

Design safety factor = 1.5 (against water pressure)

Design load : $1.5 \times 30 - 30 = 15$ kips / ft

Assume 25' anchor spacing

$25 \times 15 / \cos 30^\circ = 433^K$ each anchor

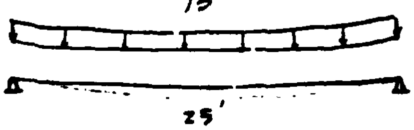
Anchor $f_y = 270$ ksi . $f_s = 162$ ksi

Design of girth beam:

$\frac{15^{K/\prime}}{8} = 1.875$

$\frac{15^{K/\prime}}{7.2} = 2.083$

10% incre....



$M = \frac{1}{8} \times 15 \times 25^2$
$= 1171.9$ K-'

6' width
$d_{min} = 29.4''$
6'x3'
$A_s = 26.4$ □"

5' width
$d_{min} = 32.2''$

Assume 15' anchor spacing (8'-0 deep)

$15' \times 15^{K/\prime} / \cos 30^\circ = 260^K$ req'd × 1.5 F.S.

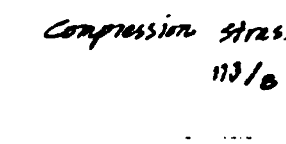
$M = \frac{1}{10} \times 15 \times 15^2$ × cos45° = 318.2^K × 1.5 = (477.3^K)

$= 337.5$

$d_{min} = 21.6''$

Beam, 8' x 3' $A_s = 2.32$ □" Use 10 - #8

check
Soil pressure



$P = 15 \times \tan 30^\circ$
$= 8.67^K$

soil pressure $= 150 \times 8 = 1200$ PSF
$+ 8670/3 \times 1 = 2890$ "
4090 PSF ←

Compression stress between beam & masonry wall:

$173/8 =$ 2165 PSF ←

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT ESTIMATE | FILE NO. | SHEET NO. 5

COMPUTED BY | CHECKED BY | DATE

Total length of perimeter :

$a = 301.5 + 23 + 1.5 = 326$

$b = 231.7 + 23 + 1.5 = 256.2$

$\therefore \ \ell = \pi(326 + 256.2)(1 + 0.25 \times 0.014 + \frac{1}{64} \times .0002)$

$= 1836'$

Total Volume of Concrete : $10 \times 3' = 30^{\square}$

$V = \cancel{36}^{30^{\square}} \times 1836' = 55{,}080/27 = 2040$ cu. yd.

$= 2448$ cu. yd.

$- 100$

$- 36 \times 110/27 = 147$

$-$ 1914 Recon. construction

$210' \times 36^{\square}/27 = 280$ cu. yd

Volume 1921 cu. yd. Say ~~2100~~ cu. yd

2040

NOTE: ORIGINAL ESTIMATE INCLUDES DEDUCTION FOR 1914 reconstruction. Add new ring at reconstructed area.

SURFACE TREATMENT

$8' \times 1836' = 14{,}688 \text{ ft}^2$

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT .... - Tendons into Sound Rock FILE NO SHEET NO 6
COMPUTED BY .... CHECKED BY .... DATE 3/6/76

(Not including 1914 reconstruction & 1921 underpinning sections)

1) Number of tendons : 108

Length (from 41' to 83') (W/15' min. embedment length into sound rock)

Total length of tendons: 6800'

1836' - 130 - 70 - 30 = 1550'/7.5' = 206.5

USE 209

2) Concrete

Girth Beam : ~~2409 Cu. Yd.~~

3) Reinforcing : 33 bars 1550

\#8 BARS : ~~30320~~ 51,150+ (excluding splices) x 2.67 #/' = ~~80955~~ # 136,560

\#6 BARS : ~~28114'~~ 3100 x 10 = 31000 x 1.502 #/' = ~~42227~~ 46,562 46,562

Total : ~~123182~~ 183,122 (say ~~62 Tons~~ 91 TONS)

Materials needed in 1914 reconstruction section only :

1914 Reconstruction (210' length) section.

Reinf. : #8 4200' x 2.67 = 11214 #

\#6 3990' x 1.502 = 5993 #

17207 say 9 Tons

Conc. : 310 Cu yd.

Tendons ; Total No. of Tendons : 15

length : 50'

Total length : 750'

Horiz 280' x 17 bars = 5,000' x 1.043 = 5215

Vert. 560 bars x 8' = 4480 x 1.043 = 4672

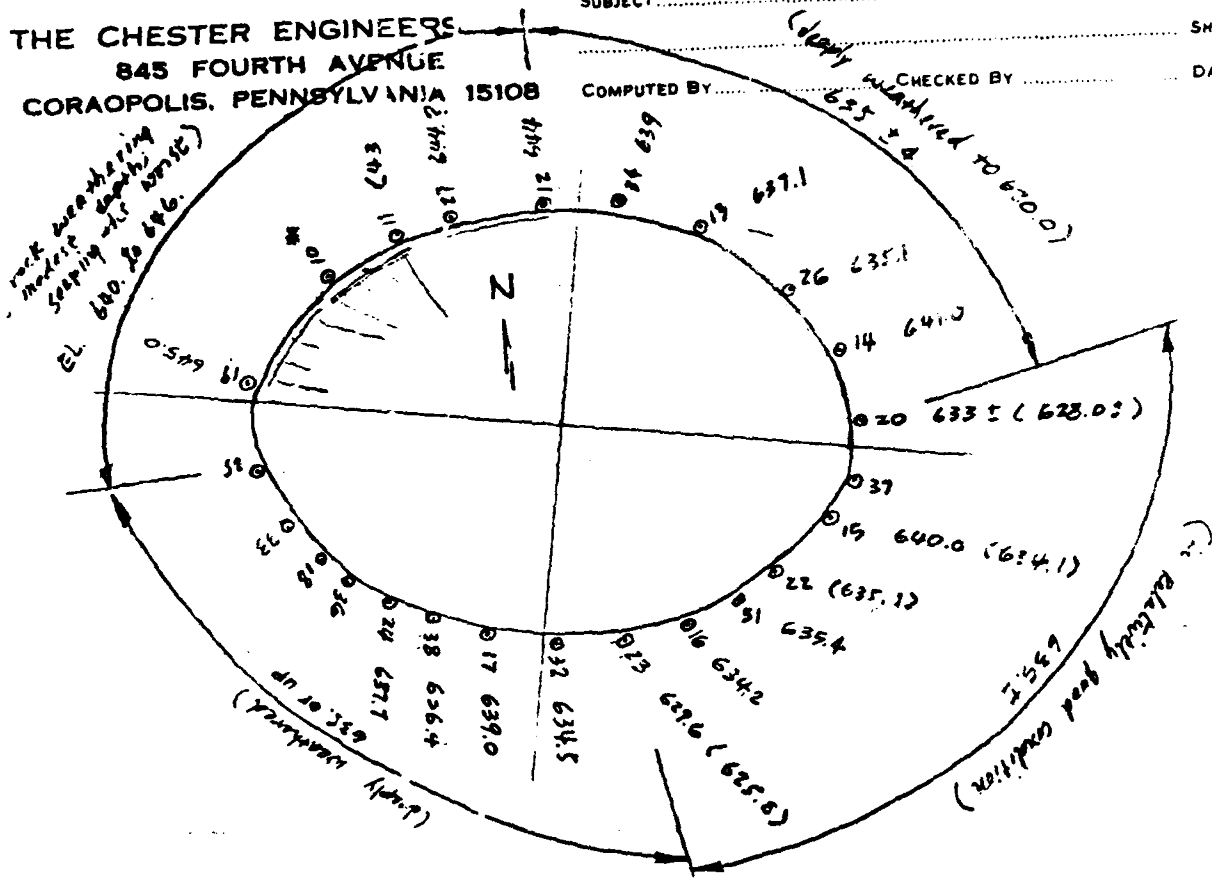
9887 #

5 TON

TOT REINF 96 TONS

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT: Nashville - Reservoir
FILE NO
SHEET NO 2
COMPUTED BY
CHECKED BY
DATE 2/3/76



xxx.xx Base of Masonry

xxx.xx Base of Concrete

xxx.xx Estimated top of rock

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT: D Ave Reservoir - Nashville
FILE NO
SHEET NO 8
COMPUTED BY .... CHECKED BY .... DATE 7/26/76

Excavation Estimates :

1) West Major axis to North minor axis .

G/EL. = 649.0

T/B EL = 6500

ℓ = 500'

$V = 500 \left[\frac{(6+3)\times 7}{2} + \frac{7\times 7}{2} \right]$

= 28000 ft^3

2) North minor axis to 1921 underpinning :

G/EL = 648.0

T/B EL = 642.0

ℓ = 410'

$V = 410 \left\{ \frac{(9+3)\times 14}{2} + \frac{14\times 14}{2} \right]$

= 74620 ft^3

3) Between 1921 underpinning & 1914 reconstruction :

G/EL = 649

T/B EL = 6440

ℓ = 60'

$V = 60 \left(\frac{(8+3)\times 13}{2} + \frac{13\times 13}{2} \right]$

= 9360 ft^3

4) Between 1914 reconstruction & 1921 underpinning

G/EL = 6470

T/B EL = 641.0

ℓ = 60'

V = 60 × [84 + 98]

= 10920 ft^3

647

64

EXCAV.

2' × 4' = 8 ft²

8 ft² × 1836 = 14,688 / 27

$\boxed{544 \text{ c.yd}}$

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT ESTIMATE
FILE NO
SHEET NO. 9
COMPUTED BY CHECKED BY DATE

5) 1921 underpinning to South minor axis

G/EL = 645

T/B EL = 640.0

l = 90'

V = 90 x 156

= 14040 ft^3

6) South minor axis to Hole No. 36

G/EL = 660

T/B EL = 645.0

l = 210'

$V = 210 \left\{ \frac{(6+3)\times 9}{2} + \frac{4\times 9}{2} \right\}$

= 17955 ft^3

7) Hole No. 36 to west major axis

G/EL = 649.0

T/B EL = 630.0

l = 200'

V = 200 x 56

= 11200 ft^3

Total excavation, 166095 ft^3 (6152 cu. yd.)

Say 6200 cu. yd

BACKFILL

6200 c.y. - 2100 c.y. = 4100 c.y. BACKFILL

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

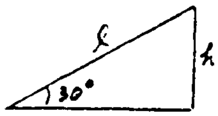
SUBJECT PRELIMINARY — FILE NO — SHEET NO 10
COMPUTED BY — CHECKED BY — DATE

Length of tendons (Estimates)

1) West major axis to North minor axis :

T/Tendon 648.0

EL of sound rock 624.0



$\ell = h/\tan 30° + 17'$ (Embedment)

$40+10 = 50$

$= 24/0.577 + 17 = 42' + 17 = 59'$

$35 \times 59 = \underline{2065'}$ ✓

2) North minor axis to 1921 underpinning :

T/Tendon 640.0

EL of rock 614.0

$\ell = 26/0.577 + 17 = 62'$

$30 \times 62 = \underline{1860'}$ ✓

3) Between 1921 underpinning and 1914 reconstruction :

T/Tendon ~~642.0~~ 652

EL of rock 624.0

$\ell = 18/0.577 + 17 = 48'$ = 40' + 10' = 50

$4 \times \cancel{48}^{50} = \underline{\cancel{192}}$ 200'

4) Between 1914 reconstruction and 1921 underpinning :

T/Tendon ~~639.0~~ 27

EL of rock 625.0

$\ell = 14/0.577 + 17 = 41'$

$4 \times 41 = \underline{\cancel{164'}}$ 200

5) 1921 underpinning to south minor axis

T/Tendon 638.0 652 617 35

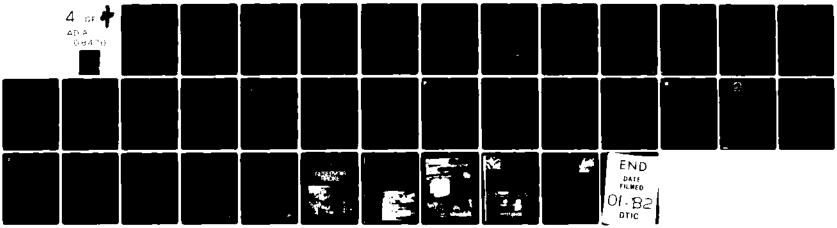
EL of rock 617.0

$\ell = 21/0.577 + 17 = 54'$

$\underline{6 \times \cancel{54}^{60} = \cancel{270'}}$ 360'

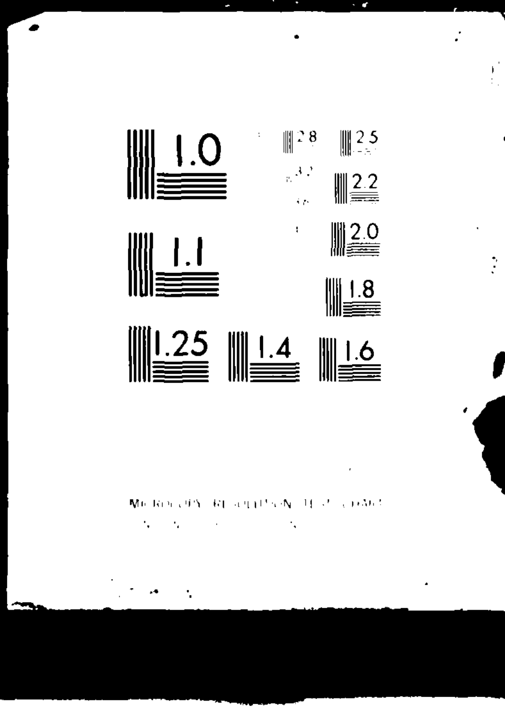
AD-A108 470 TENNESSEE STATE DEPT OF CONSERVATION NASHVILLE DIV O--ETC F/G 13/13
NATIONAL PROGRAM OF INSPECTION OF NON-FEDERAL DAMS, TENNESSEE. --ETC(U)
SEP 81 P F BLUHM DACW62-81-C-0056
UNCLASSIFIED NL

4 OF 4
AD A
108470



4 OF 4

AD A
108470



THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT PRELIMINARY — FILE NO. — SHEET NO. 11
COMPUTED BY — CHECKED BY — DATE

6) South minor axis to Hole No. 36

T/Tendon 643.0

El. of rock 615.0

$l = 28/_{0.577} + 17 = 66'$

$15 \times 66 = \underline{990}$ = 990'

7) Hole No. 36 to west major axis

T/Tendon 648.0

El. of rock 610.0

$l = 38/_{0.577} + 17 = 83'$

$14 \times 83 = \underline{1162'}$

$\frac{652}{610}$
$42 + 15$

$(52')(14) = 728'$

Total length of tendons: 6757', Say 6800' ←

spacing @15' 6313'
× 2
spacing at 7'-6" 12626

THE CHESTER ENGINEERS
845 FOURTH AVENUE
CORAOPOLIS, PENNSYLVANIA 15108

SUBJECT: PRELIMINARY FILE NO
SHEET NO 12
COMPUTED BY CHECKED BY DATE 3/16/76

TEST HOLE NO	GROUND ELEV.	BASE OF MASONRY	ELEV OF SOUND ROCK	TOP OF CANTI. SUPPORT	FREE STANDING HEIGHT
19	649.0	645.	628.	642.	3.
10	647.3	642.	627.	638.	4
11	647.9	644.	[illegible]	639	5.
27	648.7	645.	624.	638.0	7
12	648.5	645.		638.0	7.
34	648.0	639.	612	627.0	12.
13	647.7	638.	614.	620.0	18.
26	647.5	635.	617.	623.0	12.
14	647.7	601.	620	626.0	15.
37 15	648.5	634.1	624.	630.	41
22	648.2	635.1	629.	635.1	-
31	647.6	635.4	615.	630.0	5.4
16	647.4	634.2	625.	626.	8.2
23	640	625.8	625	622.	3.8
32	644.5	634.5	618.	620.	14.5
17	647.2	639.0	619.	635.	19
38	644.7	636.4	613	632.	
24	645.3	640.	612.	628	16
36	645.6	640.	610.	626	18.
18	648.7	644	610.	632	26.
33	646.6	643.1	[illegible]	628.	13.1
25	649.8	645.5	628.	628	16.5

Region — West Major Axis & Hole No 12

Top of Caisson	6450	Distance L = 427'	
Elev. of top support	6380	Ave. Length of Caissons = 20'	
Elev. of Sound Rock	627.0	No. of Caissons = 72	
Cantilever hight	70'	Σ Length of Caisson = 1440'	

Assume Caissons @ 6'-0

$M = 6 \times \frac{15}{7} \times 7^2 \times 0.5 = 315$

Try W 30 x 108 (Fy = 50 ksi)

$\frac{l}{r_T} = \frac{14 \times 12}{2.64} = 63.6$ $F_b = \frac{1000}{20 \times 2.64} = 27.1$

$\sqrt{\frac{510 \times 10^3 \times 1.0}{50}} = 101.0$

$\sqrt{\frac{102 \times 10^3 \times 1.0}{50}} = 45.2$

$F_b = \left\{ \frac{2}{3} - \frac{50\ (63.6)^2}{1530 \times 10^3} \right\} 50$

$= 26.7$ ksi

$F_a = \frac{315 \times 12}{300} = 12.6$

Try W 27 x 76

$\frac{l}{r_T} = \frac{14 \times 12}{2.32} = 72.4$

$F_b = 24.8$

$F_a = \frac{315 \times 12}{176} = 21.5 < F_b$ O.K.

Use W 24 x 76

MAX. passive pressure = 3.3 KSF

(30" ø conc. around steel pier)

Region: Hole No. 12 to Hole No. 20

Top of Caisson . 639.0
Elev. of Top support : 625.0
Elev. of Sound Rock : 614.0
Cantilever height 14.0'
$\frac{l}{r_T} = 123.9$ Use W30 × 132 $F_b = 11.1$
Max. spacing $2 \times 11.1 \times 380 / (12 \times 14 \times 15) = \underline{3.3'}$

Distance L = 445'
Ave. length of Caissons = 27'
No of Caissons = 135'
Σ length of " = 3645'

Max. passive pressure = 1.8 KSF

Region : Hole No. 20 to Hole No. 23

Top of Casson : 634.0
Elev. of Top support : 627.0
Elev. of Sound Rock : 622.0
Cantilever height 12'
Use W 30×132 Caissons @ 5'-0 o.c.
$F_b = 15.0$
$F_a = 14.2 < F_b$ O.K.

Distance L = 336'
Ave length of Caissons = 18'
No of Caissons = 68
Σ length of " = 1224
(embedment length of caissons into sound rock = 6'-0)
(O.K.)

Region, Hole No 23 to Hole No. 25

Top of Caisson : 640.50
Elev. of Top support : 628.00
Elev. of Sound Rock : 613.70
Cantilever height 12.50'

Distance L = 540'
Ave. length of Caissons = 27.0'
No. of Caissons = 136
Σ length of " = 3672'

Assume Caissons @ 4'-0 o.c.
Use W30×132

$$M = 4 \times 0.5 \times \frac{15}{12.5} \times 12.5^2$$

$$= 375'$$

$$\frac{l}{r_T} = \frac{2 \times 12.5 \times 12}{2.72} = 110.3$$

$F_b = 14.0$ ← or $F_b = 13.9$

$$F_a = \frac{375 \times 12}{380} = 11.8 < F_b \quad \text{O.K.}$$

Max. passive pressure = 2.0 KSF

Caissons

W 30 x 132

271 of 27'-0 caissons

Length = 7317'

68 of 18'-0 caissons

+) Length = 1224'

8541'

Weight = 564 Tons

W 14 x 76

72 of 20'-0 cassions

Length = 1440'

weight = 55 Tons

Total length = 9981'

Total Weight = 619 Tons

Conc. Caps : (3'x 2')

1) Conc. $3 \times 2 \times 1836 = 11016\ ft^3$ (say 450 Tons) 408 c.y.

2) Reinf.

#9

$763 \times 4 = 3052' \times 3.4\ \#/' = 10377\ \#$

#7

$1746 \times 2 = 3492'$

$983 \times 2 = 1966'$

$\Big\}\ 5458' \times 2.044\ \#/' = 11156\ \#$

Total Weight 21533#

Say 12 Tons

Conc. arround Steel piers : (30" ϕ)

$\frac{\pi}{4} \times \left(\frac{30}{12}\right)^2 \times 9981 = 48994\ ft^3$ (say 2000 Tons)

Total Conc. needed **2450 Cu. Yd.**

STRESSTEEL

October 17, 1977

Hardaway Construction Company
615 Main Street
Nashville, Tennessee 37201

Attention: Martin Keenan

Subject: Our WO 77-47
Eighth Avenue Reservoir

Gentlemen:

This will confirm our telephone conversation of October 14, in which we discussed the results of the meeting held on October 13, at the office of the Chester Engineers in Coraopolis.

For those tendons which apparently have failed in the anchor zone for reasons which are unable to be determined at the present time, the following course of action is suggested:

1. Increase the anchor force in the two immediately adjacent tendons to 0.7 f'_s or 222.6 kips.

2. Abandon the tendon which has failed.

For those tendons which show evidence of blockage in the coupler shield, with the exception of the tendon at Station 1+87.5, the following course of action is recommended:

1. Tension to 160 kips.

2. Continue tensioning for the additional strain to compensate for wedge seating loss.

3. Seat the wedges.

This will result in the tendon which is anchored at the force prescribed by the specification.

In the case of the tendon at 1+87.5, there is, apparently, nothing to be lost by an attempt to clear the blocked coupler shield and thereby restore it to full intended service. If that can be done, then the tendon at 1+87.5 and the tendon at Station 1+72.5 can be anchored at 222.6 kips and the tendon at Station 1+80 abandoned. However, if it is not possible to restore the tendon at Station 1+87.5 to full intended service, a new tendon will have to be added somewhere between Station 1+80 and 1+87.5.

You, as general contractor, should prepare a written proposal and submit it to both Geologic Associates and the Chester Engineers for approval. Based on the verbal agreement reached in Chester's office between Ray Washburn, Ed Weinheimer, Tom Chin, and myself, Chester will approve such a proposal.

Very truly yours,

Ronald J. Bonomo
Vice President - Operations

RJB: bjf

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

October 24, 1977

The Chester Engineers
2501 Hillsboro Road
Nashville, TN 37212

Attention: Mr. Richard Duncan

Gentlemen:

Re: Repairs to
Eighth Avenue Reservoir
Project No. 76-WC-47A

With reference to the above work and a letter to you from Hardaway Construction Company, Inc., dated October 21, 1977, we reply herewith.

As background, you will recall that tendons on Line 'B' at Stations 1+42.5, 1+80, and 3+77.5 failed during the tensioning process. After discussions involving the contractor, his supplier (Stressteel) as well as you, Mr. Fithian and Mr. Washburn, the following operation was authorized verbally.

Owing to the fact that the concrete ring wall was designed to safety withstand tensioning forces applied on a spacing of fifteen feet, the tendons on Line 'B', at Stations, 1+35, 1+50, 1+72.5, 1+87.5, 3+70, and 3+85 were to be re-tensioned to 222.6 Kips, each. This anchor force is equivalent to 0.7 f'_s.

BRANCHES: ROUTE 33, McBRIDE LANE, KNOXVILLE, TN 37922 615-966-9761
2712 FT. CAMPBELL BLVD., HOPKINSVILLE, KY 42240 502-886-0721
P. O. BOX 988, KINGSPORT, TN 37662 615-246-4491

Further, tendons on Line 'B', at Stations, 1+87.5, 1+95, 2+10, 2+17.5 and 2+22.5 did not elongate during tensioning, when subjected to the proof stress of 230 Kips, to the extension calculated. However, graphs resulting from the plotting of incremental loading indicated that all, except for Station 1+87.5, could be safely tensioned to the working load of 160 Kips.

The tendons at Sta. 1+87.5 'B' were a special case in that, in order to preclude installing a new set nearby, it would be necessary to free these sufficiently to permit a working stress of 222.6 Kips. Subsequently, Hardaway devised a special reamer and the tendon pair at Station 1+87.5 'B' was repaired.

On Friday, October 21, the following operations were completed:

a) The tendons at Stations 1+72.5 'B' and 1+87.5 'B' were restressed and anchored at 222.6 Kips working load. This operation rectified the failure at Station 1+80 'B'.

b) Tendon pairs at Stations 1+35 'B' and 1+50 'B' were stressed to a working load of 222.6 Kips and anchored to replace those at Station 1+42.5 'B'.

c) A similar operation anchored the tendons at Station 3+70 'B' and 3+85 'B' at 222.6 Kips to span the tendons which failed at Station 3+77.5 'B'.

d) The tendon pairs at Stations 1+95 'B', 2+10 'B', 2+17.5 'B' and 2+22.5 'B' were tensioned and anchored at their design working load of 160 Kips.

These operations satisfactorily completed the tendon anchorage system for the East Basin. One pair of tendons at Station 2+37.5 'A' should not be cut off until they are checked for lift-off pressure one more time. Incidentally, nine tendon pairs were rechecked for their ability to maintain working load prior to cut-off. Of these, only the tensons noted above will require further checking. All remaining tendons may be cut-off and sealed with grout.

Respectfully,

GEOLOGIC ASSOCIATES, INC.

R T Throckmorton Jr.

R. T. Throckmorton, Jr., P. E.

RTT/mr

Copy: Mr. K. R. Harrington
Mr. Martin Keenan
Mr. Ray Washburn

ADDITIONAL EXPLORATION
EIGHTH AVENUE RESERVOIR
NASHVILLE, TENNESSEE

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

November 23, 1977

The Chester Engineers
2501 Hillsboro Road
Nashville, Tennessee 37212

Attention: Mr. Richard Duncan

Gentlemen:

Re: Additional Exploration
Eighth Avenue Reservoir
Nashville, Tennessee

With reference to the above site, we have completed the drilling of five additional exploratory holes in the foundation for the east basin. The subsurface data is shown on the appended drawings.

When construction work began in and under the east basin we noted that the initial drilling for tendons indicated that poor subsurface conditions existed beneath the 1921 underpinning near the south axis. Subsequently, cored holes angled at 45° showed that poor quality concrete (1921) and deeply weathered rock underlay this area. Further exploration eastward showed that the southern extremity of the 1914 reconstruction underground is several feet eastward of the contact visible on the wall. Exploration of the remainder of this latter section indicates that it is in acceptable condition. We point out that to a degree the present exploration had been planned - for as

a reliable means of establishing the "overlap" necessary for various segments of the ring wall - tendon system.

Based on the data we conclude that it is prudent to extend the underpinning entirely across the 1921 reconstruction near the south axis and thence into the 1914 area for a distance of about sixty feet. Parenthetically, the new data indicates that the 1912 failure occurred, initially, within the major fault zone which crosses the wall near Hole 31.

On Monday, November 21st, we discussed this matter thoroughly with Mr. Duncan and Mr. Weinheimer, and after examining the data and the core you agreed with our interpretation. Accordingly, we conferred with Mr. E. C. Hughes, Hardaway Construction Company, and computed the stationing and lengths of tendons required to perform this work. A copy of these data is enclosed.

Owing to the season of the year and various other construction-related constraints, we recommend that contract 76-WC-47A be quickly modified to permit this change.

Yours very truly,

GEOLOGIC ASSOCIATES, INC.

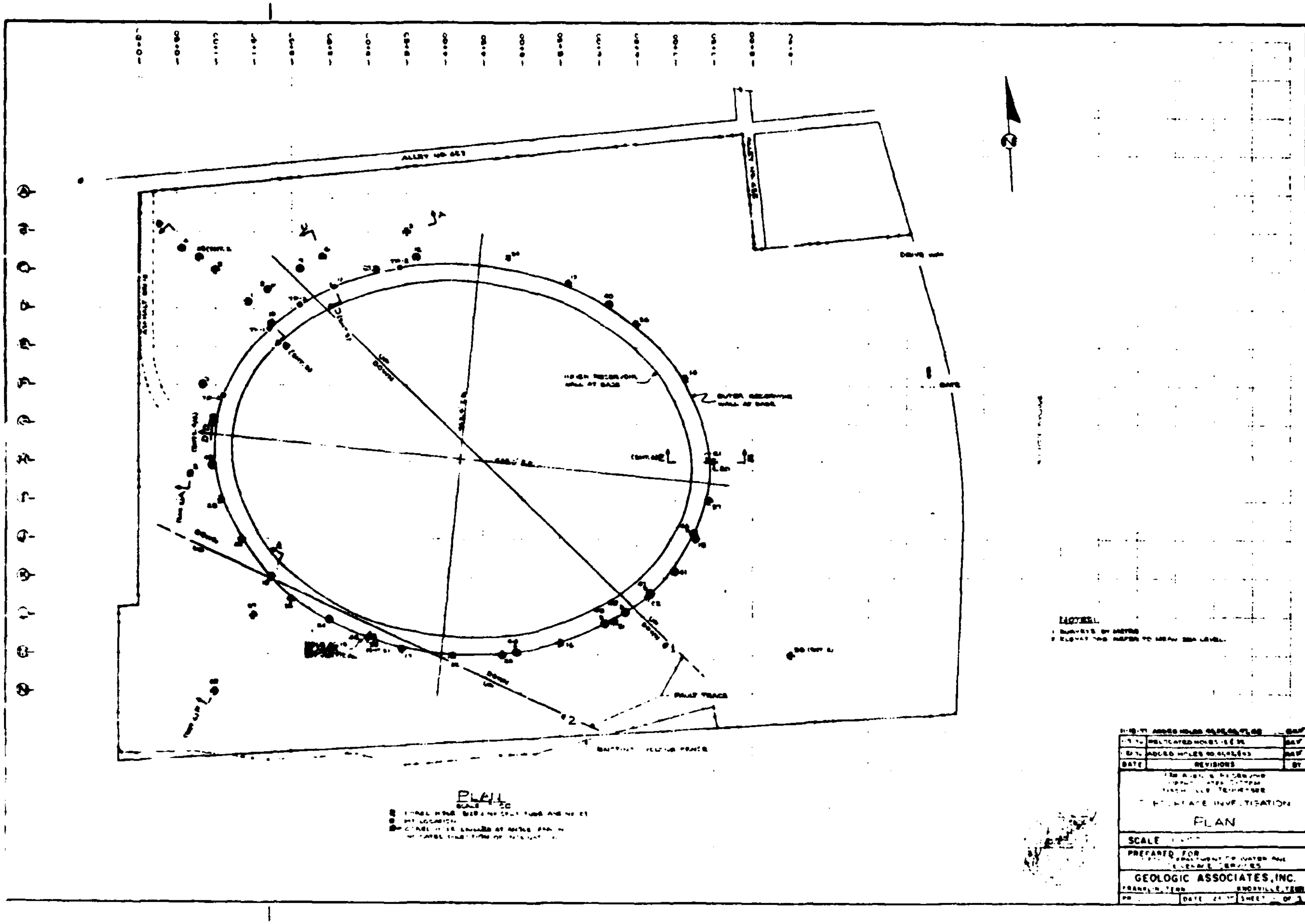
R. T. Throckmorton, Jr., P. E.

RTT/mr
Copies: Mr. K. R. Harrington
Mr. Ed Weinheimer
Enclosures: Data, 1
Drawings, 3

FOR CHESTER ENGS BY OA. FRANKLIN, TENNESSEE KNOXVILLE, TENNESSEE DATE 11-21-77

TENDONS LINE "B"

STATION	ELEV. BOTTOM ANCHOR	ANCHOR LENGTH FROM 653.0	REMARKS
5+35	—	—	CONST. JOINT
5+37.5	—	—	TENDON ON SITE
5+45.0	612.0	58.0	NEW
5+52.5	612.5	57.3	NEW
5+60.0	612.5	57.3	NEW
5+67.5	613.0	56.6	NEW
5+75	613.0	56.6	NEW
5+82.5	613.0	56.6	NEW
5+90.0	613.0	56.6	NEW
5+97.5	613.0	56.6	FOR PLANNED
6+05.0	613.0	56.6	FOR PLANNED
6+12.5	612.0	58.0	FOR PLANNED
6+20.0	611.0	59.4	FOR PLANNED
6+27.5	610.5	60.1	FOR PLANNED
6+30.0	—	—	CONST. JOINT
6+32.5	610.0	60.8	FOR PLANNED
6+40.0	609.5	61.5	FOR PLANNED
6+47.5	609.0	62.2	FOR PLANNED
6+55.0	608.0	63.6	FOR PLANNED
6+62.5	608.0	63.6	FOR PLANNED
6+70.0	608.0	63.6	NEW
6+77.5	608.0	63.6	NEW
6+85.0	608.0	63.6	NEW
6+92.5	608.0	63.6	NEW
7+00.0	608.0	63.6	NEW
7+07.5	608.0	63.6	NEW
7+15.0	608.0	63.6	NEW
7+22.5	608.0	63.6	– USE TENDON FROM 4+50 "A"
7+30.0	608.0	63.6	– USE TENDON FROM 4+57.5 "A"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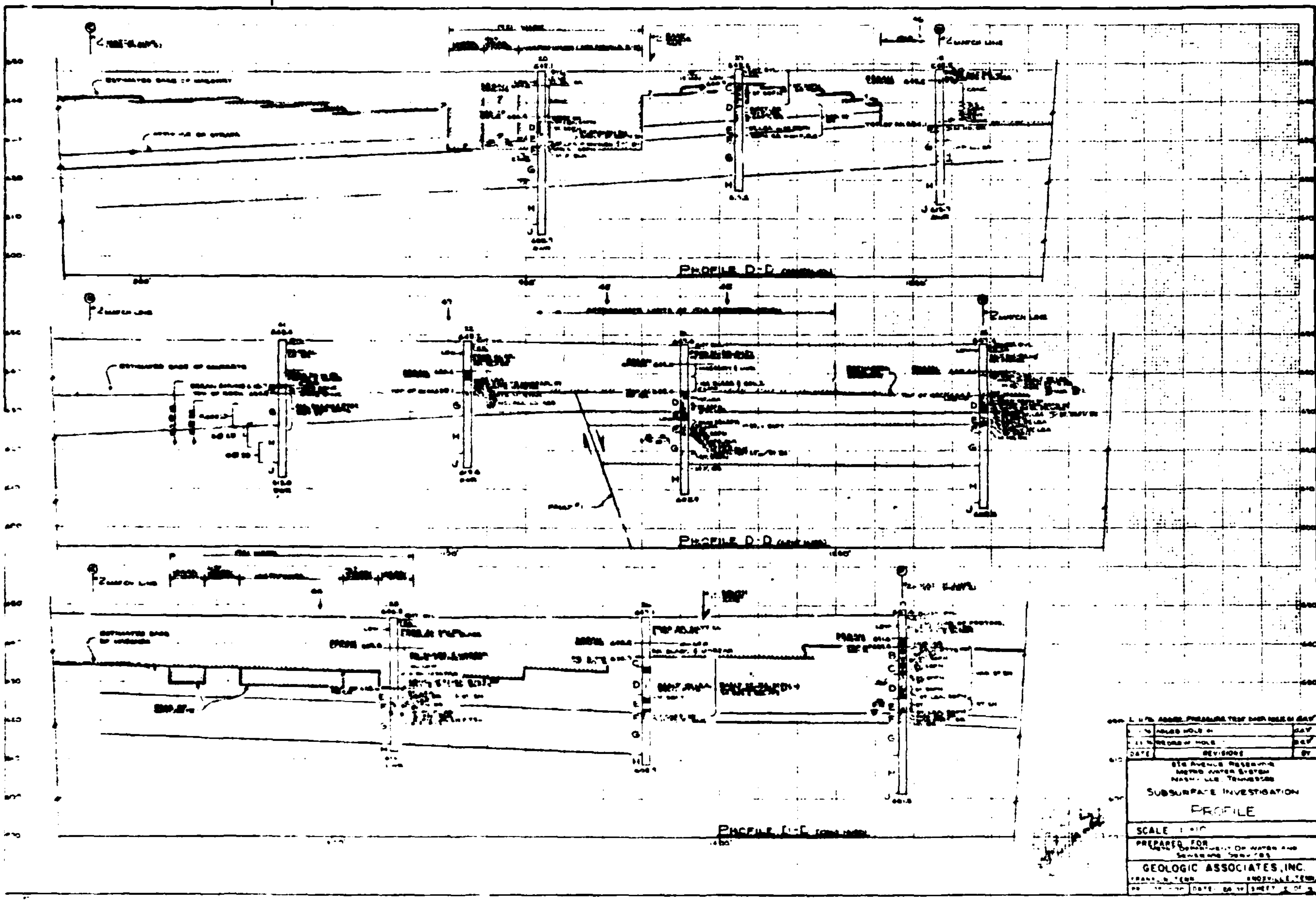
SHEET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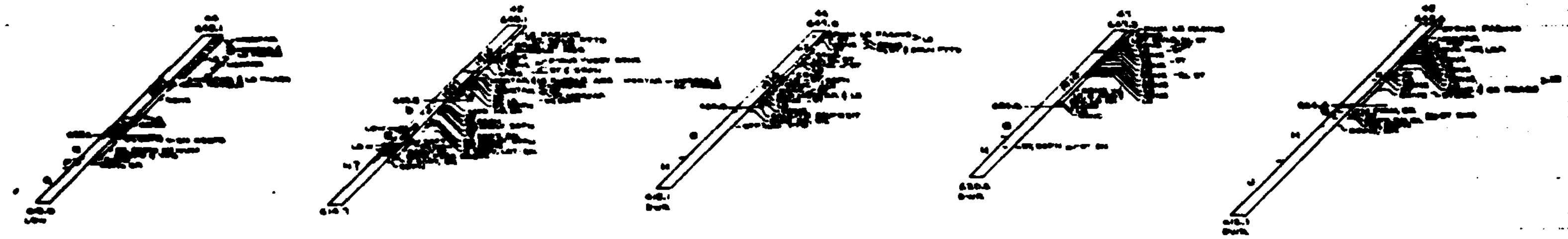


DATE	REVISIONS	BY

Nashville, Tennessee

SUBSURFACE INVESTIGATION

LOGS

SCALE AS SHOWN

PREPARED FOR
Sewerage Services

GEOLOGIC ASSOCIATES, INC.

FRANKLIN, TENN.

PROJ 76 070 | DATE | SHEET 1 OF 1

DRAWN BY [illegible]

GEOLOGIC ASSOCIATES, INC.
GEOLOGISTS AND ENGINEERS

REPLY TO:
P. O. BOX 668
FRANKLIN, TN. 37064
615-794-3596

February 28, 1978

RECEIVED
MAR 1 1978
WATER & SEWERAGE SERVICES
Nashville, Tennessee 37201

The Chester Engineers
2501 Hillsboro Road
Nashville, TN 37212

Attention: Mr. Richard Duncan

Gentlemen:

Re: Geotechnical Services,
Instrumentation for Eighth Avenue Reservoir
Project No. 78-310

During the thorough study made of subsurface conditions at the reservoir, numerous holes were core drilled around the perimeter of the wall. Subsequently, ten of these holes were utilized during the installation of an inclinometer monitoring system. Parenthetically, four other holes some distance away from the reservoir at various points comprise the remainder of the system. As you will recall, the inclinometer system was our only method of monitoring the condition of the subgrade of the reservoir prior to completion of the extensometer array. Since that time the inclinometers have been more or less on "standby" because the extensometers permit more precise monitoring. However, they have been checked from time to time.

The location of these drill holes and the special casing grouted in them is such that they are just inside or outside of the toe of the concrete ringwall. Their location(s) posed no problem until it was decided to cover the concrete ringwall with an earth blanket. Contract 76-WC-47A has now been modified to provide for this effort. Consequently, in order to continue to use the inclinometers the riser pipes must be extended through the earth cover. Furthermore, in order to protect the special plastic casing, it will be necessary to concrete 3" ф iron pipe around it.

We have carefully considered the matter and believe that the Owner would be well-advised to authorize this additional expenditure, considering the considerable investment already made in installing and monitoring these features. Moreover, it is wise to have this system available on a standby basis for the foreseeable future.

The task of raising the pipes and performing the peripheral work involved must be done by our forces with the cooperation of the contractor. Based on our current schedule of fees and the material costs, we estimate the cost of this effort at $3,800.

Please authorize this work as soon as possible, because the contractor intends to place the earth berm as soon as weather permits.

Yours truly,

GEOLOGIC ASSOCIATES, INC.

Ray Throckmorton

R. T. Throckmorton, Jr., P. E.

RTT/mr

Copy: Mr. K. R. Harrington

Geologic Associates, Inc. GEOLOGISTS AND ENGINEERS

9/14/78 - Bill Brodie, Bobby Williams, Mike ... Richard Duncan Review & provide comments

September 18, 1978

RECEIVED

W. F. Baird assignment

Metropolitan Government of Nashville
and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, Tennessee
Attention: Mr. K. R. Harrington, Director

Re: 8th Avenue Reservoir
Continuing Monitoring
of The Extensometer System

Gentlemen:

As you know, the terminal monitor has now been relocated to its permanent position in the operations building. The system is functioning pretty well; however, we are obtaining erratic readings, they seem to fluctuate on a daily basis from several of the units. We suspect that some moisture in connections at the junction boxes may be the cause. We have ordered packets of dessicant which may solve the problem. In the interim, we don't see any purpose in having Service Construction check the system. On the other hand, we prefer not to release them until the system functions correctly, as it did prior to the rewiring necessitated by the move.

While the extensometer system has proved somewhat "temperamental" with reference to electrical surges, etc., we believe that it has been of utmost value during the reconstruction. If nothing else,

PO Box 008 - Franklin, Tn 37064 - 615 794 3590
Franklin, Tenn. - Knoxville, Tenn. - Kingsport, Tenn. - Hopkinsville, Ky

It permitted us to "feel comfortable" with reference to stability during installation and tensioning of the tendons.

Now that this reconstruction effort, which at least skirted the fringes of the state of the art, is complete, we recommend that the system continue to be monitored, recorded, and assessed on a daily basis for perhaps a year. If you wish, we will continue to utilize the remote terminal to record several readings daily via the data line. These data will continue to be plotted in the permanent record. Incidentally these latter records now comprise several rolls of drawings. At some point we expect to deliver all of these to you for your files. Further, continuing monitoring will entail our maintaining the system in working order, as well as in occasionally visiting the site in order to make mechanical readings of the extensometers which are used to verify the accuracy of the electronic system. As usual, we will charge for our services based on the time utilized for various skill levels. Will you wish us to continue to bill through Chester Engineers, or bill the Department directly?

Metropolitan Government of Nashville
and Davidson County
Page Three
September 18, 1978

Additionally, we think it pertinent to provide your on-site personnel with a brief, simple orientation as to what their duties are in reference to the system and the terminal monitor in particular. Owing to the rotation of shifts, it will probably be necessary for us to go over this with them on a more or less individual basis. Perhaps Mr. Williams will tell us how he'd like to go about this aspect.

Finally, before long we expect to provide drawings pertaining to the as-built retention system. Obviously, these need to go into your permanent files. If you wish us to provide Chester Engineers with a set, please notify us.

At your convenience, we can discuss any aspects of these matters.

Yours very truly,

GEOLOGIC ASSOCIATES, INC.

Ray Throckmorton

R. T. Throckmorton, Jr.

RTT/slt

ENVIRONMENTAL
ENGINEERS & PLANNERS
845 Fourth Avenue
Coraopolis, Pa. 15108
Phone: 412/771-4380
Phone: 412/262-1036

Vur

File

THE CHESTER ENGINEERS

October 28, 1977

Ref. No. 0698

RECEIVED
NOV 1 1977
WATER & SEWERAGE SERVICES
Nashville, Tennessee 372..

Hardaway Construction Company, Inc.
615 Main Street
P. O. Box 60464
Nashville, Tennessee 37206

Attention: Mr. Martin O. Keenan

Re: 8th Avenue 50 MG Reservoir
Project No. 76-WC-47A

Gentlemen:

We have reviewed the letters of Geologic Associates, Inc. regarding repairs to this station dated October 24, 1977, and also a copy of the letter from Stressteel Company to your office dated October 17, 1977, which summarizes and discusses the conclusions which were reached in this office at a meeting with the Stressteel people on October 13, 1977, and later confirmed by telephone conversation with Ray Throckmorton of Geologic Associates Inc.

The contents of these letters verify the discussions and decisions which were reached in the above meetings and conversations and also verify the fact that verbal approval to proceed with the action designated is hereby confirmed.

Very truly yours,

THE CHESTER ENGINEERS

R. B. Washburn

RBW/aed

cc: R. Duncan
K. Harrington
T. Soster
R. Throckmorton

10/1/77 – copy to Bill Brook
Bobby Williams, Gae Johnson

(615) 242-9692

HARDAWAY CONSTRUCTION COMPANY, INC

615 MAIN STREET/P.O. BOX 60464/NASHVILLE, TENNESSEE/37206

October 21, 1977

The Chester Engineers
845 Fourth Avenue
Coraopolis, Pennsylvania 15108

Attention: Mr. Ray Washburn

Re: 8th Avenue 50 MG Reservoir
Project No. 76MC47A

Dear Mr. Washburn:

The purpose of this letter is to reconfirm the decision points reached at the conclusion of a meeting held last Thursday, October 13, 1977, in your office with Mr. Ronald J. Bonomo of Stressteel Corporation, Mr. Ed Weinheimer, Mr. Tom Chin and yourself. As Mr. Bonomo representated our concern at that meeting he has transmitted information concerning the decisions reached and as discussed herein. We also wish to recommend the course of action developed at that meeting as the solution to discrepancies which have occurred during the stressing of seven each tendons at the above referenced project.

For those tendons which apparently have failed in the anchor zone for reasons which are unable to be determined at the present time we recommend the following action:

A. Increase the anchor force in the two immediately adjacent tendons to $0.7f'_s$ or 222.6 kips.

B. Abandon the tendon which has failed. Since the two tendons in question which fall in this category are at station 1+80 and 3+77.5 tendons at station 1+72.5, 1+87.5, 3+70 and 3+85 will be stressed and anchored at 222.6 kips. The tendons at 1+80 and 3+77.5 will be abandoned. The tendon at 1+37.5 is one which evidenced a blockage coupler shield. That blockage has been removed and will now be loaded as indicated above.

For those tendons which indicated a blockage of the coupler shield at stations 1+95, 2+10, 2+17.5 and 2+22.5 the following course of action is recommended:

A. Tension the rock anchor to a working load of 160 kips.

B. Continue with tensioning for the additional strain to compensate for wedge seating loss.

C. Seat the wedges. This action will result in these tendons being anchored at the working force prescribed by the contract specifications.

Your immediate review and response to these recommendations will be greatly appreciated.

Very truly yours,

HARDAWAY CONSTRUCTION COMPANY, INC.

Martin O. Keenan
Project Manager
Construction Group I

MOK/dg

cc: Mr. R. D. Duncan - Chester Engineers
Mr. R. T. Throckmorton - Geologic Associates
Mr. R. J. Bonomo -Stressteel Corporation
Mr. B. D. Grover

File: 1179-08-00-04

VWF
For Comment?
[illegible]
Is it really necessary?
[illegible]

ENVIRONMENTAL
ENGINEERS & PLANNERS
[illegible] HILLSBORO ROAD
[illegible], TENNESSEE 37212

THE CHESTER ENGINEERS

5

December 6, 1978

Ref. No. 698-35A

RECEIVED
DEC [illegible] 1978

Mr. K.R. Harrington, Director
Metropolitan Department of Water
and Sewerage Services
301 Stahlman Building
Nashville, Tennessee 37201

Dear Mr. Harrington:

Project No. 76-WC-47
8th Ave. Reservoir (Instrumentation & Monitoring)

Reference is made to Geologic Associates' letter to you, dated November 28, 1978, concerning the continuation of monitoring of the subject instrumentation system.

We concur in the scope of the contract as described in the letter, and recommend that the Metropolitan Government enter into such a contract. We do note, however, that the applicable unit prices in the fee schedule are approximately 43% higher than the ones Geologic Associates used on the original reservoir work.

Please feel free to call on us if we can be of additional assistance.

Sincerely yours,

THE CHESTER ENGINEERS, INC.

Richard D. Duncan

Richard D. Duncan, P.E.
Director, Nashville Office

RDD/js

cc: T.C.E., Pittsburgh
(John Kane)

12/7/78 – Copy to Bill Brooks, Bobby Williams
Gene Johnson, Mike Patterson
Bill & Bobby review and advise

[illegible]

Geologic Associates, Inc. GEOLOGISTS AND ENGINEERS

11/29/78 - Copy to Bill Brook, Buddy Williams
Mike Parkman, Richard Duncan for review and comment

Copy to Gene Johnson

November 28, 1978

RECEIVED
NOV 29 1978
WATER & SEWERAGE SERVICES
Nashville, Tennessee 37201

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, TN 37201

Attention: Mr. K. R. Harrington, Jr., Director

Gentlemen:

Re: Continuation of Monitoring
of Instrumentation
Eighth Avenue Reservoir
Nashville, Tennessee

With reference to the above and our various conversations regarding it, we herewith submit our proposal.

We have delayed submitting a proposal until we could evaluate a couple of months of post-reconstruction operations in order to see what may be required.

Initially, upon installation of the system and throughout the reconstruction; we not only kept voluminous records, but spent a good deal of effort in tabulating and plotting the data. This effort was valuable in aiding us in our assessment of the safety of the operation. The time consumed by draftsmen was expensive, also.

25th Anniversary

P.O. Box 888 - Franklin, Tn 37064 - 615-794-3596
Franklin, Tenn. - Knoxville, Tenn. - Kingsport, Tenn. - Hopkinsville, Ky.

In the light of the apparently "nominal" current performance of the reservoir, we now recommend that regular monitoring continue, but that the complex tabulation of data be kept to a minimum. We expect to retain the printed tapes of the readings which are made several times daily and to tabulate the readings for ready reference. However, the plotting, which is most time consuming, has been discontinued; it can be done at any time if necessary. Further, obviously, a complex monitoring system such as this will require some "maintenance". We have already had to work on one of the sensors in an attempt to overcome some erratic readings.

Therefore, in light of the foregoing we estimate that the cost of our services will average about $600 per month; however, you will be billed only for the amount of time actually spent. If this proposal is acceptable, please issue an agreement of whatever sort is appropriate based on the applicable unit prices in the enclosed current fee schedule. Parenthetically, total charges for November are $484.

We appreciate this opportunity to be of continuing service.

Yours very truly,

GEOLOGIC ASSOCIATES, INC.

Ray Throckmorton

R. T. Throckmorton, Jr.

RTT/mr

Enclosure: Fee Schedule

GA

Geologic Associates, Inc. GEOLOGISTS AND ENGINEERS

GAJ please discuss with me

February 26, 1980

RECEIVED
FEB 27 1980
METROPOLITAN DEPARTMENT OF WATER & SEWERAGE SERVICES

FIND OUT EXP. DATE & WRITE LETTER FOR CONE.

Metropolitan Government of
Nashville and Davidson County
Department of Water and Sewerage Services
Stahlman Building
Nashville, TN 37201

Attention: Mr. Lester Williams

Gentlemen:

Re: Monitoring,
Eighth Avenue Reservoir

We have now completed another year of monitoring the extensometer system at this facility.

In the light of the continuing satisfactory operation of the reservoir, we recommend that our contract be terminated and that regularly scheduled monitoring cease.

If you wish, we can remove and store the master terminal console and the accessory monitoring components for future use or sale. We point out that after the master terminal is removed, the extensometers can be monitored on an individual basis with the portable read-out, should the need arise. We will await your instructions.

We appreciate our participation in this project and look forward to many more years of service as your geotechnical consultants.

Cordially,

GEOLOGIC ASSOCIATES, INC.

Ray Throckmorton

R. T. Throckmorton, Jr.

RTT/mr

GA
Founded 1953

P O Box 668 · Franklin, TN 37064 · (615) 794-3595
Franklin - Knoxville - Kingsport, TN

THE DAY THE RESERVOIR BROKE

By Max York

Kirkpatrick Hill is green and quiet now.

Signs warn visitors they are not welcome. But if you are old enough to remember, you have earned the right to ignore signs, at least for a quick walk up a hill.

Horace Hime walks close enough to see that his memories aren't playing tricks on him. The signs of the violence of that night are there. There is a section of the east wall of the reservoir in which the stonework does not match the rest of the structure.

"Our back yard was over there," he says, pointing to a fenced in area behind an office building on the downtown side of the reservoir on Eighth Avenue. "Our house was where that building is now.

"Down there where the liquor store is, there was a grocery store. Across the street there where the swim place is now, there was a Methodist Church in that same building. The rest of the street held nice, neat houses."

It was a good place to live. He and his twin brother, Hardin, walked to Fall School [illegible] down the street, passing a series of billboards on the way.

"We were seven years old," he says. "You started school at seven then."

The school building still stands. At least that hasn't changed. It no longer is a school, but it's there.

Some time after that night, Edwin and Ada Hime took Horace and his six brothers and moved to another neighborhood, on 11th Avenue South.

He went to Clemmons School. When he was 22 and a photographer, Horace Hime went away to Chicago and shot photographs for ads that would appear in such magazines as *Life*, *Liberty*, *Look* and the *Saturday Evening Post*. He was in Chicago for half a century.

"I came back last May," he says.

His third day home he had a heart attack. He has been a heart patient since. But there has been time to look up old friends.

"As far as I can tell, there isn't another person except for my twin brother, still alive who lived on that street that night. If there are, they would have to have been children not much older than we were."

Even among the friends he knew in high school, his calls found mostly widows.

If there are others, it is doubtful they have forgotten that night.

It came after a night of campaign rallies. The nation was destined to go to the polls in the daylight hours of that day, November 5, 1912, and elect Woodrow Wilson president of the United States.

But at that hour, there was darkness. Any person who wasn't home in bed at 10 minutes past midnight certainly was up to no good. One man coming home about 11 o'clock later recalled he thought he heard a trickle of water. He considered telling the watchman at the reservoir. Instead, he went to bed.

Horace Hime was in bed and had been for a long time.

"I had never heard a tornado," Hime says. "I still haven't. But I imagine that is what a tornado sounds like. There was a terrible roar. We all ran out on the porch.

"I remember I was in my night clothes. They were something like long johns. You stepped into them and they covered you from your neck to your feet. They were made of flannel. I wasn't cold.

"There was a river of water. It must have been nine feet deep, and it was going right by the house. The road leading up to the reservoir was right next to our house. The road was cut deep into the hill. The water was rushing down that road. We were up high. So we were safe."

Hime remembers he wasn't afraid. Looking back, he imagines his parents and his older brothers must have been.

"At seven years of age, it was all exciting," he says. "I was getting a big kick out of it. I'm sure the older ones realized the seriousness of it.

"There was no thought of running away. There was no place to go. We were just fortunate that road bed was so deep."

Others were not so fortunate. The man who came and heard the trickle of water, Hillman Cheatham, later told reporters he had just gotten into bed when he heard a deafening crash. Then came the water. It slammed into the house and came inside but did little damage. A horse and buggy were swept from his barn. He lived at 1506 Eighth Ave. S.

At 1502, T.M. Heffrey and his wife and child were swept from their house, floating on their bed. At 1504, W.O. Arzinger went to see what was going on and was knocked down by the water before he could get to the door.

Along Eighth Avenue and on Lynwood houses were being swept from their foundation. The water swept everything in its path. Stones from the reservoir, some weighing tons, were washed onto both streets.

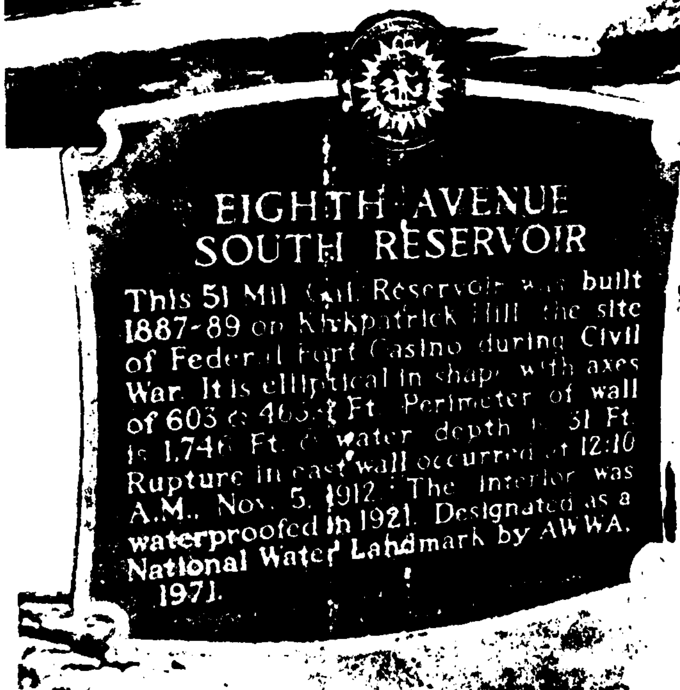
Firemen came to rescue people from the water. Despite the late hour, hundreds of people wanted to do what they could for their neighbors. They searched for bodies, too, knowing that night must have held death.

"I had forgotten until my brother reminded me," Hime says, "but a little later one of our parents, he seems to think it was our mother, took us around the back way up to the house up on the reservoir's west side. Our parent banged on the door. The man came to the door and was told that the reservoir had broken. He went away and came back and said, 'My God! You're right!'

"It was quite a shock. The reservoir has two sections. The east section had

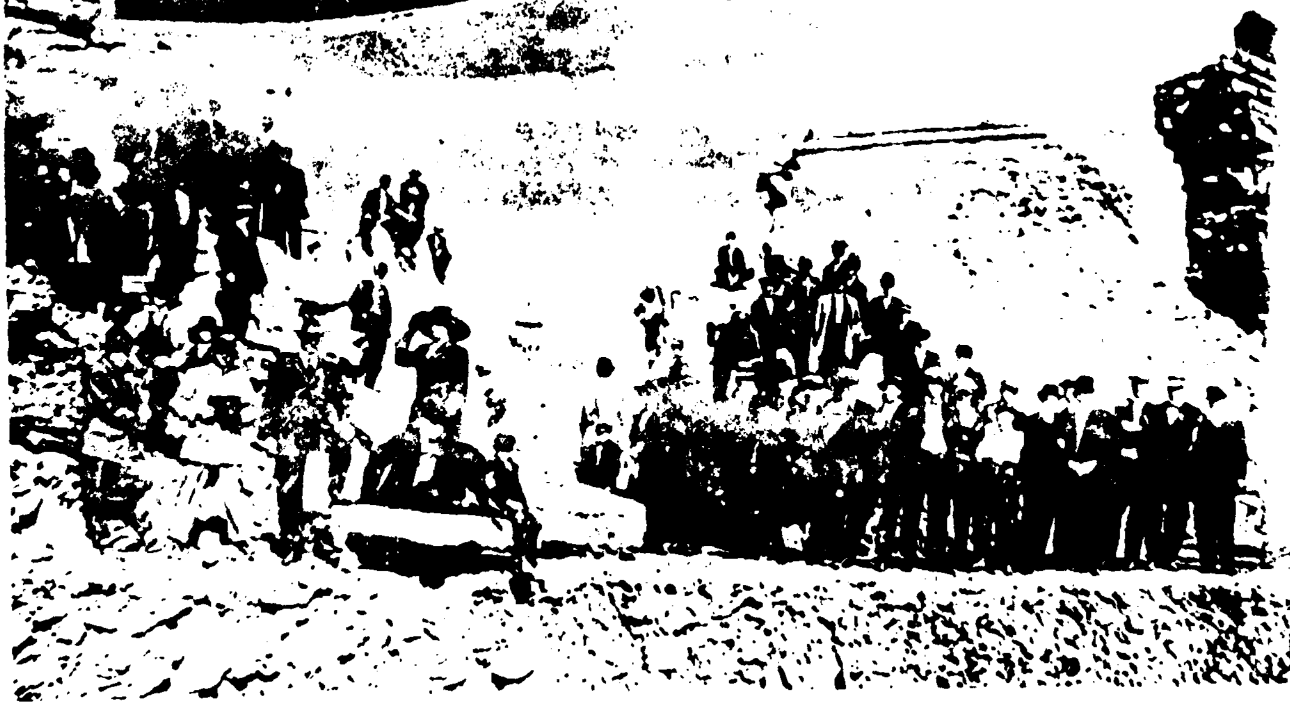
In the background, the area where the wall was repaired is still visible, and Horace Hime remembers the night it happened.

Hime lived in a house where the building below now is located. The water came down that road from the reservoir, forming a nine-foot-deep river.



The reservoir also is listed by the National Register of Historic Places.

The morning after the accident, the disaster area was something of a tourist attraction, and everybody was ready to



When this was taken in 1913, repair work was underway

broken. The west section was still holding fine."

The watchmen on duty, M. Sinnot and Ben Hall had a different story for reporters. They said they heard the crash and ran out of the office and saw the water level falling rapidly.

The break had spilled 25 million gallons of water But the other half of the reservoir held an equal amount, enough water to last the city for four days.

The next day an estimated 25,000 people came to see the destruction. Dead chickens, clothing, furniture and debris were everywhere.

"I remember going out the next morning," Hime says. "There were a lot of people in the street and in the areas where the houses had been destroyed, but there weren't many people up by the reservoir."

The thing he remembers most about that day is the kind of thing a boy of seven would find most important.

"There were large puddles of water the next morning," Hime says. "They must have been 25 or 30 feet from the reservoir wall. Most of the rest of the water was gone. These puddles were full of catfish.

"They were flopping around. Nobody was paying any attention to them."

Looking back with a boy's eyes, the fish were about 18 inches long and wider than a man's hands and there were a lot of them.

"That was the first time I knew there were fish in the reservoir. I don't know how they got there. I've heard there still are some in there. I don't know what happened to those fish that day. They were still there when I left. I imagine they ended up on somebody's supper table."

The crowds were enormous. Eighth Avenue was littered with huge rocks, soil and debris.

"The streetcar from downtown ran only as far as the reservoir," Hime remembers. "People going on out would have to get off, walk three blocks and catch another car that would turn up at Douglas Corner and go over to Tenth Avenue."

Some of the boys in the neighborhood took a piece of the uprooted street car track and put it on somebody's front porch, Hime says. It also was a time for boyish pranks.

The cleanup went on for some time.

"The dirt and stone were at least three feet deep in the street," he says.

"The next day a photographer from Wiles Studio was back out there selling eight by ten photographs taken the day before. It was a real mess in that neighborhood."

There was much destruction, but fortunately nobody was killed. That was something of a miracle. In fact, nobody was seriously hurt.

The old reservoir still serves Nashville. Except for that one night, it has served the city well since 1889, the year it was completed on the hill that was the site of Fort Casino, a federal stronghold during the Civil War

The reservoir now is covered with a nylon tarpaulin. Nobody considered it unsafe, but in recent years, engineers began a project to anchor the limestone structure to solid rock. It has never been safer.

It's place in history is assured, too. Recently the reservoir was added to the list of the National Register of Historic Places.

"We were just fortunate that road was cut deep in that hill," Hime says. "If it hadn't been, I might not be here."

There is little of Nashville that reminds him of the city he knew as a boy, but the hill and the reservoir do.

"Those stone gateposts there at the entrance were here back then," he remembers. "I used to play all over that hill."

Ringwall and tendons halt leaks in 90-year-old landmark reservoir

Nashville's historic stone wall reservoir is getting a new lease on life with the installation of a ringwall and tendon system to bring the structure up to modern safety standards. A national waterworks landmark, the 90-year-old reservoir has been plagued by recurring leaks and was the victim of a major wall failure in 1912 (ENGINEERING NEWS 11/14/12 p. 922).

A leak detected in one of the reservoir's two 25-million-gal basins after a refurbishing in 1975, which included the addition of a floating vinyl cover to comply with U.S. Environmental Protection Agency (EPA) standards, led Nashville officials to hire Geologic Associates, Inc. (GA), Franklin, Tenn., to appraise the structural stability of the walls.

Initial explorations in the area of the leak indicated that subsurface materials around the reservoir were of poor quality and that "the wall undoubtably exists in a precarious state of stability," says Raymond T. Throckmorton, a partner in GA.

The consultant also found that two inactive faults pass under the reservoir. The larger one exits the reservoir near the area of the 1912 failure and the worst leaks of 1975.

Extensive monitoring. Mindful of the earlier wall failure, the age of the structure and a lack of original building plans, GA's crews approached the reservoir cautiously in their early explorations and studies. To check on movement within the walls and underlying bedrock, 20 extensometers, or stress gages, were driven into the wall and 70 ft into the bedrock. The monitoring system will remain in place after repairs are completed.

"As far as we know, no one has a very good handle on how far [the wall] could move before it failed. We think it would move a good deal and then go with a snap," warns Throckmorton.

Assessing seismic risks is also difficult. In one report to city and county officials, Throckmorton notes that Nashville sits in a Zone 1 seismic risk area and is sandwiched by Zone 2 contours (Zone 0 being the lowest seismic risk and Zone 3 the highest).

Enlargement and reconstruction of the structure was not feasible because it would mean taking the reservoir out of service. The eventual choice was a ringwall to girdle most of the reservoir and post-tensioning tendons for stability.

The 8-ft-high, 3-ft-thick reinforced concrete ringwall is wrapped around the base of the reservoir. Two $1^{1}/_{8}$-in. steel tendons are inserted through the ringwall and the reservoir wall into the bedrock at a 45 deg angle at 226 points around the wall. Spacing ranges from 5 to 7.5 ft depending on ringwall construction joints. Depth of the tendon anchors ranges from

Bars stressed through ringwall into rock

5 to 7.5 ft depending on construction joints. Depth of the tendon anchors ranges from 28 to 7[illegible] [illegible] tendons is stressed to 239,000 lb [illegible] 80 [illegible] of bar tensile strength, before final stressing at roughly 169,000 lb.

A 122-ft section repaired after the 1912 break and a 55-ft section strengthened in 1921 were omitted from the repairs.

The ringwall and tendon system was designed by Chester Engineers, Coraopolis, Pa., consultant for Nashville's waterworks for more than 50 years. Repairs were are being done by Hardaway Construction Co., Inc., Nashville, under a $679,500 contract. Work which began in May, 1977, is expected to be completed this month.

Additional remedial repairs were undertaken to waterproof the reservoir's basins to halt further seepage and deterioration of subsurface soils. While one 25-million-gal basin continued operation, the other was drained and cleaned. The floating vinyl cover was recut, cleaned and converted to liners to accomplish the waterproofing.

The reservoir is currently operating without covers under a variance granted by EPA until repairs are done. The elliptical-50-million-gal reservoir was completed in 1889 (ENGINEERING NEWS 6/16/1888 p. 484) and has a major axis of 603 ft and a minor axis of 463 ft. The 1,837 ft circumference is constructed of stone masonry walls about 34 ft high, 2[illegible] ft wide at the base and 8 ft wide on top.

On Nov. 4, 1912, after residents of the area complained to city officials about leakage, a 150-ft section of wall slid about 30 ft off its foundation, causing extensive property damage but no loss of life. It was repaired at a cost of over $100,000 and returned to service in 1914 (ENGINEERING NEWS 2/16/13 p. 278 and 5/16/[illegible] p. 849).

Engineer probes reservoir history in old articles

Trying to appraise the stability of Nashville's [illegible] stone wall reservoir required that Raymond T. Throckmorton of Geologic Associates become a historian.

Armed with the original design drawings and project engineer's notes and drawings from the archives of Chester Engineers on some additional repairs made in 1920, Throckmorton still had nothing to indicate the extent to which actual construction followed the design.

Although he stumbled on the references to ENGINEERING NEWS – which merged with ENGINEERING RECORD in 1917 and became ENR – late in the game, "They were invaluable in helping us understand what had been done in the reconstruction process," says Throckmorton.

Throckmorton used four articles from ENGINEERING NEWS. The first, published in June, 1888, detailed the reservoir construction process. The other three articles published in 1912, 1913 and 1915 described the wall failure and covered the reconstruction.

His detective work disclosed that "numerous 30-ft railroad rails" had gone into a buttress built as part of the wall repairs in the failed section. The rails would have been a mystery without the ENGINEERING NEWS articles. Acquaintance with their general location and purpose permitted those doing current remedial work to accurately predict their location and miss them with the tendon holes, says Throckmorton.

Throckmorton also tracked down several photographs of the 1912 failure. Enlargements showed large portions of the failed wall remained intact, leading the consultant to conclude they slid outward because of poor foundation materials, the same conclusion reached in 1912 and reported in ENGINEERING NEWS.